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ANALYSIS OF STRUCTURAL INSTRUMENTATION DATA FOR LOCK AND DAM NO. 1, RED RIVER

by

Rex C. Donahey

School of Civil Engineering
Oklahoma State University
Stillwater, Oklahoma 74074

and

Robert L. Hall

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

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This report contains an evaluation of data obtained from instrumentation of Lock and Dam No. 1 on the Red River and corresponding structural analyses. This work was undertaken because of unexpected heavy silt buildup during the spring of 1985. The computer program, CUFRAM, was used for the structural analyses. The lock monolith L-10 was used for the comparisons in this report. Measured soil pressures were used as input pressures to CUFRAM. Member forces obtained from the analyses were compared with forces calculated using the measured strains in the reinforcing steel. The calculated moments were shown to have relatively good agreement with moments from the measured strain data. However, axial forces obtained using the same procedures were significantly different. The reason for this discrepancy was discussed.

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PREFACE

This summary of the structural instrumentation data and comparative structural analyses of the U-frame lock structure for Lock and Dam No. 1 on the Red River was prepared for the US Army Engineer District, Vicksburg. The instrumentation data were obtained from Mr. Jon M. Andersen and Mrs. Wipawi Vanadit-Ellis of the Geotechnical Laboratory, US Army Engineer Waterways Experiment Station (WES). The program CUFRAM, developed by Dr. William Dawkins, Information Technology Laboratory, WES, was used to provide the structural analyses. The structural dimensions and material properties were provided by the US Army Engineer District, Vicksburg.

The CUFRAM analyses and strain-gage analyses were conducted by Dr. Rex C. Donahey, Assistant Professor of Civil Engineering, Oklahoma State University. The conclusions and preparation of the report were completed by Dr. Donahey and Dr. Robert L. Hall, Research Structural Engineer, Structural Mechanics Division (SMD), Structures Laboratory (SL), WES. Dr. Hall also managed and coordinated the work with the Vicksburg District and WES under the general supervision of Messrs. Bryant Mather, Chief, SL, James T. Ballard, Assistant Chief, SL, and under the direct supervision of Dr. Jimmy P. Balsara, Chief, SMD.

COL Dwayne G. Lee, EN, was the Commander and Director of WES when this report was prepared. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
foot-pounds (force)	1.355818	joules
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
kips (force) per foot	14.5939	kilonewtons per metre
kips per square inch	6.894757	megapascals
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches per foot	21.1666	square centimetres per metre

ANALYSIS OF STRUCTURAL INSTRUMENTATION DATA FOR LOCK AND DAM NO. 1, RED RIVER

PART I: INTRODUCTION

Background

1. The lock structure for Lock and Dam No. 1 of the Red River Waterway is a U-frame, consisting of 18 monoliths. A typical cross section for a chamber monolith is shown in Figure 1. The chamber is 84 ft* wide and 49.5 ft deep. The base slab is 12 ft thick, and the chamber walls are 8 ft thick at the base, tapering to 4 ft thick at an elevation of 58.5 ft above the base slab. Two 12- by 12-ft culverts are located on each side of the chamber.

2. During construction, extensive instrumentation was installed both in and around the lock structure and the dam. Lock instrumentation included strain gages on the reinforcing steel, soil pressure meters, piezometers, and inclinometers. All instruments were monitored both during and after construction. A description of the instruments and a summary of the instrumentation data are contained in Anderson and Vanadit-Ellis (1987).

3. In March and April 1985, flooding of the Red River resulted in the buildup of approximately 10 ft of silt on the riverside of the lock. The silt was removed before the river elevation returned to normal.

4. It is anticipated that silt buildup will continue to be a problem with Lock and Dam No. 1. Because of this, at least three alternative management schemes are being considered for the structure: (a) make no changes to the lock structure, but continue to remove silt buildup as it occurs; (b) modify the lock structure to include a wall between the river and the lock; and (c) made no changes to the lock structure and allow silt buildup.

5. The initial two schemes will involve considerable expense. The first will require a continuing presence of expensive dredging equipment at the lock structure; the second will have a substantial initial cost. Also, in the event of a large flood, the second alternative will make it almost impossible to remove any silt which may overflow the protective wall.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

6. Although it will be less expensive than the first two alternatives, the third scheme requires that the structure be analyzed for the possibly high silt loadings which will occur. While finite element analyses of lock structures have been found to provide very good correlation with instrumentation data (Clough and Duncan 1969), such analyses continue to be quite expensive. Relatively inexpensive analyses can be conducted using the stiffness method of matrix analysis. To date, however, the results of stiffness method analyses have not been compared with instrumentation data. Clearly, if the Red River Lock structure is to be subjected to extreme silt loadings, it must be analyzed for those loadings, and confidence in the analysis procedure must be established. The large amount of instrumentation on the Red River Lock structure makes it a reasonable choice for comparison of stiffness method analyses with the observed behavior of the lock.

Previous Work

7. The initial applications of the U-frame concept for lock structures were the Port Allen and the Old River Locks. Both were instrumented with soil pressure meters, settlement points, and piezometers (Kaufman and Sherman 1964). A series of finite element analyses of the Port Allen and Old River Locks were subsequently conducted by Clough and Duncan (1969). Two methods of analysis, a gravity turn-on analysis and an incremental analysis, were used. The deflections and soil pressures obtained from the analyses were compared with the instrumentation data.

8. The study indicated that a gravity turn-on analysis provides reasonable agreement with pressure meter data when gravity is applied only to the lock structure and the backfill. The study also indicated that an incremental analysis can provide excellent agreement with measured soil pressures.

9. Clough and Duncan also evaluated the effects of seasonal temperature changes on the lock structures. Temperature variations were shown to produce wall deflections of more than two times those produced by filling the lock chamber with water. Measured and calculated soil pressures were shown to increase during the summer months, as the lock walls moved outward into the backfill.

10. More recently, a two-dimensional gravity turn-on analysis program for U-frame structures, CUFRAM (Dawkins 1987), was developed for the Corps of

Engineers. The program is configured to allow rapid investigation of multiple load cases and provides moments, shears, and axial loads as output. Lock structures are modeled using beam elements, with rigid links used at member intersections to account for the large increases in member stiffness at the joints. The length of the rigid links is user adjustable.

11. Results of CUFRAM analyses were compared with the results of finite element analyses using isoparametric elements.* The comparisons showed that CUFRAM gives relatively good agreement with finite element method (FEM) analyses. It was also shown that CUFRAM analyses using rigid links which extend from member center lines to the face of members provide the best agreement with FEM results.

Object and Scope

12. The possibility of continued silt buildup at Lock and Dam No. 1 makes it necessary to analyze the structure for overload. The program CUFRAM will allow relatively inexpensive analyses; however, confidence in CUFRAM analyses must be established.

13. This report contains an evaluation of data obtained from a portion of the instrumentation in lock Monolith L-10 of Lock and Dam No. 1. Measured soil pressures at Monolith L-10 are used as input soil pressures in CUFRAM. Member forces obtained using CUFRAM are compared with forces calculated using the measured strains in the reinforcing steel. The moments obtained with CUFRAM are shown to have relatively good agreement with moments calculated from the measured strain data; however, axial forces obtained using the two procedures differ significantly. Reasons for the discrepancy are discussed.

14. Finally, CUFRAM is used to calculate member forces under a worst-case silt load. Critical member forces are compared with allowable member forces from a strength analysis of the structure.

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* Robert L. Hall, unpublished investigation data, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

PART II: ANALYSIS

15. Lock Monolith L-10 was selected for all analyses. Monolith L-10 is a chamber monolith located between sta 3+88L and 4+32L. It contains extensive instrumentation, with a total of 34 soil pressure meters (SPM) and 57 reinforcing steel strain gages (RSG).

16. Soil pressure distributions at Monolith L-10 were obtained from SPM data. These distributions were used, in turn, as input for CUFRAM analyses of the lock under different operating conditions. Strain data obtained from the RSG's were used to obtain estimates of the axial loads and moments in the lock members.

Measured Data

Soil pressure meters

17. Soil pressure meters were installed in Monolith L-10 at two stations: SPM's 25-31 were installed at sta 4+10L, while SPM's 32-59 were installed at sta 4+12L. The approximate locations of the SPM's on the lock structure are shown in Figure 2.

18. Soil pressures obtained on six dates were used for CUFRAM input: 9-22-83, 12-13-83, 3-16-85, 9-19-85, and 1-29-86. The 9-22-83 and 12-13-83 dates are of interest because the backfill was complete, but the lock chambers had not yet been flooded. The 3-16-85 date is of interest because the readings were taken during the heavy flooding of the Red River. The 6-11-85 date is typical of normal, warm-weather conditions. The 9-19-85 date is considered because the readings were taken when the lock had been dewatered for repairs. Finally, the 1-29-86 date is considered because it is one of the more recent dates for which data are available and is typical of normal, cool weather conditions.

19. Pressure data from the SPM's are presented in Table 1 for the six dates considered. The piezometric head in the subgrade (obtained from piezometer 104) is also given for each date.

20. The pressure data obtained for the base slab are plotted as a function of location is Figures 3-5. The pressure data for the riverside wall of the lock are shown in Figures 6-8, while the pressure data for the landside of the lock are shown in Figures 9-11. All values shown are total pressures.

Strain gage data

21. Reinforcing steel strain gages were installed at sta 4+10L in monolith L-10. RSG's 1-6 were installed in the riverside stem wall, RSG's 7-35 were installed in the riverside culvert walls, and RSG's 36-57 were installed in the base slab (Figure 12). Strain data from the RSG's are presented in Table 2 for the six dates described in the previous section.

22. Plots of the RSG data for the stem wall and the base slab were made to establish that the gages were functional. Figure 13 shows the strain in the stem wall reinforcement as a function of the elevation of the gages. Figure 14 shows the strain in the steel in the base slab as a function of distance from the longitudinal center line of the chamber.

23. The strain data from the stem wall show that the inner face is in compression, while the outer face tends to be in tension. The increasing difference in the strain values with decreasing elevation shows that the strain data reflect the increasing moment (increasing curvature) near the base of the wall. Also, the plot shows that the strains are relatively smooth functions of position, indicating gages were installed properly and are functional.

24. The strain data from the base slab show that the base slab is in a hogging moment over the entire width of the lock. That is, the top of the slab is in tension, while the bottom of the slab tends to be in compression. This agrees with what one would expect for a slab-on-grade with large concentrated loads (due to the walls) at both ends. As with the wall strains, the strains are relatively smooth functions of position.

CUFRAM Analyses

CUFRAM input

25. The member sizes used in the CUFRAM analyses were taken from drawings 7/46 and 7/49 of the project plans. A typical cross section is shown in Figure 15.

26. SPM data for the six selected dates were used to provide input for CUFRAM. CUFRAM allows the user to manually specify input pressure distributions for both sides of the lock, as well as the base slab. For all analyses, the total pressure data obtained from the SPM's were used.

27. Lateral earth pressures were obtained from SPM's 25, 26, 27, 54,

31, and 33 on the riverside of the lock and from SPM's 58, 57, 30, and 51 on the landside of the lock. With the exception of the 3-16-85 analysis, the point of zero pressure was set at an elevation of 31 ft as referenced to the National Geodetic Vertical Datum (NGVD) (42 ft from the top of the base slab). The point of zero pressure for the 3-16-85 analysis was set at 43.3 ft, which was the peak flood stage for the river. Lateral earth pressures in the regions between SPM's 27 and 54 and between SPM's 57 and 30 were obtained by linearly extrapolating the pressures from above and below to an elevation of 4.5 ft. Lateral earth pressures below SPM's 33 and 51 were obtained by linear extrapolation to an elevation of -17 ft. At points where the measured pressures were less than the piezometrix head, the piezometric head was used as input.

28. Vertical earth pressures were assumed to be linear functions of the soil depth. A constant unit weight of 120 pcf was used in all analyses. Although shear stress in the soil is allowed as CUFRAM input, no data were available for the structure. Shear stresses were, therefore, assumed to be zero.

29. Base slab pressures were obtained from SPM's 34-50. Pressures at sta 0+68D and -0+68D were considered to be equal to the piezometric head at piezometer P104.

30. Examples of the input pressure distributions for the CUFRAM input are given in Figures 16-24. Figures 16-18 show the distributions for 12-13-83, Figures 19-21 show the distributions for 3-16-85, and Figures 22-24 show the distributions for 6-11-85.

31. The water elevation in the chamber was estimated for each date considered. In most cases, the elevation was set equal to the piezometric head obtained from P104. The exception was for the 12-13-83 analysis, in which the water elevation was assumed to be zero.

CUFRAM results

32. CUFRAM allows two general types of analysis: an equilibrium analysis and a frame analysis. The equilibrium analysis checks vertical, horizontal, and moment equilibrium for the lock structure, considering the input pressures, water elevations, and member weights. The frame analysis provides member forces, including moment, shear, and axiai load.

33. Equilibrium analyses. Each equilibrium served as a check of both the measured soil pressure distributions and the assumed internal water

elevations. CUFRAM has the capability to automatically adjust the base pressures to maintain vertical and moment equilibrium. The required adjustments were relatively low. For example, the ratio of the input pressure to the adjusted pressure at the same point varied from 0.988 to 1.052 for the 3-16-85 analysis and from 1.015 to 1.091 for the 6-11-85 analysis. The relatively low adjustment required in the base pressure distributions is an indication that the measured pressures are correct.

34. CUFRAM can automatically adjust for horizontal equilibrium either by removing unbalanced forces through base friction or through horizontal shear in the base slab. In the current study, all unbalanced horizontal forces were removed through friction. For all six dates, the unbalanced forces were relatively large. For example, the force imbalances were 5.9 kips/ft, 29.5 kips/ft, and 10 kips/ft for the 12-13-83, 3-16-85, and 6-11-85 analyses, respectively. In all cases, the force imbalance was toward the landside of the lock, reflecting the higher pressures measured in the riverside SPM's. The very high unbalanced force for 3-16-85 reflects the silting conditions which existed at the time. It is apparent the lock structure remained relatively stationary during this period. Had the structure moved, the landside pressure distribution would have increased as the structure shifted.

35. Frame analyses. The U-frame structure is modeled using seven beam elements on each side of the chamber center line (Figure 25). Beam elements M1 and M2 represent the base slab. Elements M3 and M4 represent the walls of the culvert, while M5 represents the top of the culvert. Finally, M6 and M7 represent the stem wall.

36. Example plots of axial load, shear, and bending moment in the lock are shown in Figures 26-34. For 12-13-83, Figures 26 and 27 show the forces in the base slab on the landside and the riverside, respectively, while Figure 28 shows the forces in the riverside stem wall. Similarly, Figures 29-31 apply to 3-16-85, and Figures 32-34 apply to 6-11-85. Axial force and bending moment for the base slab and stem wall are compared with analyses of strain gage data in the section entitled "Strain Gage Analyses."

Strain Gage Analyses

37. RSG data were used to calculate the axial load and bending moment

at selected points within lock Monolith L-10. Two sections were selected in the base slab: 16.5 ft (RSG's 42 and 43) and 42 ft (RSG's 36 and 37) from the lock center line (Figure 12). The section at 16.5 ft from the center line is the first section with complete strain data (two working gage installations) near the point of maximum curvature. The section at 42 ft is near the stem wall. One section was selected in the stem wall, at an elevation of 11 ft (RSG's 5 and 6).

38. Reinforcing steel areas and locations were obtained from drawing 7/50 of the project plans.

Force calculation procedure

39. Internal moment and axial force calculations were based on the following simplifying assumptions:

a. Strain compatibility exists between the reinforcing steel and the concrete, as well as between adjacent concrete lifts.

b. Compression stress in the concrete f'_c is a second-order function of the compression strain in the concrete ε_c (Figure 35) and is given by

$$\frac{f_c}{f'_c} = 2\left(\frac{\varepsilon_c}{\varepsilon_o}\right) - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2 \qquad (1)$$

where

ε_o = strain at f'_c (Hognestad 1951)

f'_c and ε_o = 3,000 psi and 0.002 in./in., respectively

c. Tensile or compressive stress in the steel reinforcement f_s is given by

$$f_s = E_s \varepsilon_s \qquad (2)$$

where

E_s = modulus of elasticity for the steel

ε_s = strain in the steel

E_s is assumed to be 29,000 ksi. It should be noted that no strains were observed to be more than 0.000485 on the dates considered.

d. The concrete carries no tensile stresses.

e. The compressive force in the concrete is calculated by integrating the stress function over the compression zone of the

member. The resultant is assumed to be applied at the centroid of the stress function.

f. Bar groups made up of multiple layers of reinforcement are lumped at the centroid of the group. Strains and stresses are considered to be uniform throughout the group. The total area of the group is multiplied by the stresses to obtain a resultant force at the group centroid.

g. The axial force within a member is the sum of the tensile and compressive forces in the reinforcing steel and the compressive force in the concrete.

h. Moments are calculated about the midline of a section.

Data summary

40. The measured strains, distances to the centroids of the reinforcing steel, and areas of the reinforcing steel groups are presented in Tables 3-5. Tables 3 and 4 contain data for the base slab at 16.5 and 42 ft from the chamber center line, respectively. Table 5 contains data for the stem wall at el 11.0 ft. All distances are referenced to the extreme compressive fiber of the concrete.

Comparison of Calculated Forces

41. Bending moments and total axial forces calculated from the strain data are presented with values from the CUFRAM analyses in Tables 6-8. The data show that the two methods do not, in general, provide the same axial forces at a section. In fact, within the base slab, the axial loads calculated using the two procedures are always of opposite sense. The bending moments do, however, have the same sense in all cases. Also, the calculated bending moments from the two methods are, in all cases, of the same order of magnitude. Bending moment calculations for the stem wall and for the base slab at the chamber center line show good agreement.

Base slab forces

42. Using the RSG data, all calculated axial forces in the base slab were tensile. The CUFRAM analyses, however, show that the base slab is always in compression. The small magnitudes of the measured strains may be one reason for the discrepancy. The RSG force calculations are very sensitive to the magnitude of the compressive strain in the concrete. It should also be noted that the base slab was placed in three lifts. The strain values obtained in the structure are therefore the result of an incremental loading. In

contrast, CUFRAM provides gravity turn-on analyses only.

43. Although the calculated axial loads do not show good agreement, the calculated bending moments do show reasonable agreement, particularly at the section near the chamber center line. The ratio of calculated moment from CUFRAM analysis to the calculated moment from the RSG data varies from 1.09 to 1.84 for the six dates considered.

Stem wall forces

44. The stem wall forces show good agreement in both calculated axial force and calculated bending moment. The axial forces for the stem wall have the same sense in all cases. Although the magnitudes of the forces calculated using the RSG data show considerable variation with time, it should be noted the data are probably very sensitive to temperature variations. Any difference in the coefficients of thermal expansion for the steel and concrete will result in significant strain differentials between the two materials as the concrete temperature fluctuates.

45. The bending moments calculated using the two methods also show reasonable agreement. The ratio of the moment from the CUFRAM analysis to the calculated moment from the RSG data varies from 0.45 to 1.35.

PART III: WORST-CASE ANALYSIS

46. The observed behavior of the lock structure shows reasonable agreement with the behavior predicted by CUFRAM analyses. This agreement indicates that CUFRAM can be used in an evaluation of the structure's performance under extreme overload.

47. In this section, an extreme silt load combined with dewatering of the chamber is discussed. Moments, shears, and axial loads at critical sections are obtained from a CUFRAM analysis and are compared with allowable values based on ACI 318-83 (American Concrete Institute (ACI) 1983).

CUFRAM Input

48. The worst-case load for the lock is assumed to consist of a saturated silt surcharge to an el of 43.3 ft on the riverside of the lock, a normal soil loading on the landside of the lock, and a complete dewatering of the lock chamber.

49. Lateral soil pressures used in the analysis are obtained from SPM data described in Part II. The riverside data are based on the 3-16-85 SPM readings, which are associated with a total backfill and silt elevation of 43.3 ft. The landside pressure distribution is assumed to be linear up to an elevation of 31 ft. The magnitude of the distribution is adjusted so that a total unbalanced horizontal load of 10 kips is attained. The unbalanced load is typical of those indicated by the SPM data over the history of the lock. In effect, the soil pressure on the landside of the lock reflects movement of the lock into the landside backfill.

50. Base pressure distributions used in the CUFRAM analysis are based on the 9-19-85 SPM data, which correspond to a dewatered lock chamber. In the CUFRAM analysis, the pressure distributions are automatically adjusted to obtain vertical force and moment equilibrium.

CUFRAM Results

51. The resultant forces obtained from 10 sections in the lock are listed in Table 9. The locations of the sections are shown in Figure 36.

52. Using the assumptions presented in Part II, the moments and axial

loads are used to calculate strains in the reinforcing steel and in the extreme compressive fiber of the concrete. The associated steel and concrete stresses are listed with the calculated strains in Table 10. The data in Table 10 show that stresses associated with axial load and moment are relatively low at all sections in the lock. The maximum tensile stress in the steel is 21.8 ksi. Although this does exceed the allowable working stress in the steel of 20 ksi, it is still well within the elastic limit for the steel. The calculated compression stresses in the concrete are all below the allowable working stress of 1.35 ksi. The shear forces from the CUFRAM analysis are presented in Table 11. Allowable shear forces, based on Equations B-2 and B-3 of ACI 318-83, are also presented in Table 11. The shear forces at three sections in the lock, B, H, and I, violate the allowable values. The ratios of calculated-to-allowable force for the three sections are 1.54, 1.20, and 1.16, respectively. It should be noted that, because of the varying depth of the top of the culvert, the shear for which section B should be evaluated, $\bar{V}$, is approximated by

$$\bar{V} = V - T \tan \theta \tag{3}$$

where

V = calculated shear

T = force in tension steel

θ = slope of top surface of culvert

Using $T = 110.5$ kips and $\tan \theta = 0.138$, V is approximately 87.9 kips. The resulting calculated-to-allowable shear ratio is, therefore, 1.31.

Discussion

53. Although the extreme loading considered above should not be considered a service load, it is a probable event in the history of the lock. The fact that some of the members will be overstressed is certainly of concern. The high shear forces calculated for three locations in the lock indicate that a more detailed evaluation of the lock should be made. For example, a reasonable value for the current concrete strength should be established. Also, correct allowable shear forces for the sections should be determined.

Although the culvert walls at sections B and H are deep beams with span-to-depth ratios of less than 5, the allowable shear forces are based on design values for long beams. Even without a more detailed evaluation, however, it should be noted that the calculated shear forces are well below the shear strength for the sections.

PART IV: CONCLUSIONS

54. The results of the CUFRAM analyses of the Red River Lock structure show reasonable agreement with observed strain gage data. The analysis of the lock structure under a worst-case loading shows that the lock may be overloaded in shear at three points in the section. Further evaluation of the lock during the worst-case loading should be considered before operating the structure during these conditions.

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Table 1

Soil-Pressure-Meter Data from 1983 to 1986

SPM No.	el ft*	Sta	Date 9-22-83	12-13-83	3-16-85	6-11-85	9-19-85	1-29-86
			Stem and Chamber Pressures, psf					
25	26.5	0+48D	350	399	1,662	597	506	973
26	18.5	0+49D	770	895	2,299	1,256	1,312	1,188
27	10.5	0+50D	1,193	1,184	3,436	1,700	1,228	1,690
28	6.8	0+53D	3,277	3,347	4,721	3,521	3,672	3,941
29	5.3	0+62D	1,445	1,321	3,090	2,139	1,708	2,075
54	0	0+68D	483	641	2,043	608	-469	22
31	-5.0	0+68D	-203	-473	2,524	1,507	-1,721	1,212
33	-16.0	0+68D	1,549	1,472	3,630	2,490	1,523	1,982
58	18.5	-0+49D	286	882	1,845	968	498	446
57	10.5	-0+50D	1,061	892	2,539	1,709	1,180	1,682
56	6.8	-0+53D	1,858	3,158	3,872	3,397	3,296	3,140
55	5.3	-0+62D	858	824	2,687	1,766	1,205	1,637
30	0	-0+68D	-226	-293	2,001	1,082	-182	631
51	-16.0	-0+68D	1,965	2,006	2,655	1,952	1,330	1,533
			Base Pressures, psf					
34	-17.5	0+66D	681	618	3,293	2,253	1,145	1,750
35	-20.3	0+58D	1,128	1,056	3,078	2,282	162	2,246
36	-23.0	0+50D	6,992	7,020	8,539	8,107	6,971	7,246
37	-23.0	0+42D	6,084	6,085	9,218	7,990	7,237	8,545
38	-23.0	0+34D	5,111	5,024	7,478	6,171	5,143	6,485
39	-23.0	0+27D	5,253	5,385	7,551	6,328	5,414	6,585
40	-23.0	0+20D	5,064	5,140	4,404	5,946	4,225	6,531
41	-23.0	0+10D	2,319	2,120	4,550	3,546	2,551	3,560
42	-23.0	0+00D	3,808	3,892	6,253	5,028	4,047	5,080
43	-23.0	-0+10D	4,777	4,948	6,914	5,912	4,976	5,338
44	-23.0	-0+20D	7,026	6,390	8,265	7,037	5,554	7,838
45	-23.0	-0+27D	6,210	6,236	7,841	6,996	6,339	6,746
46	-23.0	-0+34D	4,756	4,727	6,503	5,756	4,923	5,640
47	-23.0	-0+42D	4,008	3,898	6,872	5,298	4,908	4,587
48	-23.0	-0+50D	5,249	5,368	7,381	6,835	5,838	6,365
49	-20.3	-0+58D	2,326	2,453	4,543	3,506	1,738	2,793
50	-17.5	-0+66D	2,823	2,796	4,548	3,248	2,292	1,773
P104 Water el =			-9.65	-8.5	43.3	23.07	-5.91	13.29

* All elevations (el) are in feet referenced to the National Geodetic Vertical Datum.

Table 2

Reinforcing Steel Strain Gage Data from 1983 to 1986

RSG No.	Dam Sta	el ft	Date 9-22-83	12-13-83	3-16-85	6-11-85	9-19-85	1-29-86
1	0+47.0D	29	-69	-17	13	-13	-29	4
2	0+43.0D	29	-90	-50	-31	-69	-77	-4
3	0+45.0D	20	40	31	90	79	44	33
4	0+43.0D	20	12	6	13	-12	-44	31
5	0+49.0D	11	38	73	104	73	60	94
6	0+43.0D	11	-82	-68	-47	*	*	*
7	0+49.0D	2	-42	-13	26	-7	-3	32
8	0+43.0D	2	-13	13	2	-10	-12	48
9	0+49.0D	-4	-68	-49	*	*	*	*
10	0+43.0D	-4	-219	-273	-254	-277	-260	-145
11	0+49.0D	-10	0	21	48	-2	6	114
12	0+43.0D	-10	-15	2	4	-35	-25	72
13	0+49.0D	-16	53	72	117	N.R.**	97	129
14	0+43.0D	-16	78	95	124	103	120	163
15	0+52.0D	0	67	96	108	60	88	179
16	0+52.0D	5	-69	-54	-4	0	-27	-37
17	0+59.0D	0	131	150	123	-19	0	143
18	0+59.0D	5	-76	-68	-20	-13	-47	-25
19	0+65.0D	0	37	54	85	58	-37	82
20	0+65.0D	4	-100	-90	-50	-46	-65	-54
21	0+46.0D	-22	27	35	92	96	79	98
22	0+52.0D	-14	-44	-13	-10	-58	-38	42
23	0+52.0D	-21	27	35	*	*	*	*
24	0+59.0D	-14	-38	-17	25	-10	19	66
25	0+59.0D	-19	-17	-6	50	54	46	73
26	0+65.0D	-14	49	64	110	101	107	125
27	0+65.0D	-17	44	58	102	106	100	115
28	0+63.0D	2	13	19	56	37	60	75
29	0+67.0D	2	-55	-51	-33	-35	-62	-40
30	0+63.0D	-4	-17	-6	13	-33	21	70
31	0+67.0D	-4	-69	-62	-23	-54	-50	-28
32	0+63.0D	-10	17	22	93	57	26	109
33	0+67.0D	-10	63	70	140	151	130	149
34	0+63.0D	-16	23	44	67	40	40	85
35	0+67.0D	-16	53	57	97	97	74	87
36	0+42.0D	-12	113	140	119	62	100	200
37	0+42.0D	-22	19	25	77	79	54	59
38	0+33.5D	-12	165	158	144	87	156	230
39	0+33.5D	-22	-13	0	*	*	*	*
40	0+25.0D	-12	223	252	215	152	219	282
41	0+25.0D	-22	-85	-85	-33	-29	-77	-61
42	0+16.5D	-12	356	383	312	254	315	415
43	0+16.5D	-22	-110	-115	-56	-50	-98	-82
44	0+08.0D	-12	306	377	*	*	*	*
45	0+08.0D	-22	-108	-115	-35	-23	-81	-46
46	0+00.0D	-12	306	421	*	*	*	*
47	0+00.0D	-22	-142	-154	-108	-108	-188	-165
48	-0+08.0D	-12	392	485	469	415	442	527
49	-0+08.0D	-22	-98	-110	*	*	*	*
50	-0+16.5D	-12	283	346	*	*	*	*
51	-0+16.5D	-22	-88	-94	-29	-17	-67	-44
52	-0+25.0D	-12	227	285	254	185	223	312
53	-0+25.0D	-22	-65	-69	-12	-35	-75	-38
54	-0+33.5D	-12	244	319	219	129	175	261
55	-0+33.5D	-22	-29	-31	-10	2	-29	-10
56	-0+42.0D	-12	92	133	129	54	79	156
57	-0+42.0D	-22	15	19	25	33	2	29

* Gage malfunction.

** N.R.--No reading recorded.

Table 3

Strain Data for Base Slab at

16.5 ft from Center Line

Date	Measured Compressive Strain, in./in.	Measured Tension Strain, in./in.
9-22-83	-110	356
12-13-83	-115	383
3-16-85	-56	312
6-11-85	-50	254
9-19-85	-98	315
1-29-86	-82	415

Note: dc = 10.0 in. = distance to comp. steel
dt = 128.6 in. = distance to tension steel
Ac = 3.81 sq in./ft of width = area of compressive steel
At = 12 sq in./ft of width = area of tension steel

Table 4

Strain Data for Base Slab at

42 ft from Center Line

Date	Measured Compressive Strain, in./in.	Measured Tension Strain, in./in.
9-22-83	19	113
12-13-83	25	140
3-16-85	77	119
6-11-85	62	79
9-19-85	54	100
1-29-86	59	200

Note: dc = 14.4 in. = distance to comp. steel
dt = 128.6 in. = distance to tension steel
Ac = 5.37 sq in./ft of width = area of compressive steel
At = 12 sq in./ft of width = area of tension steel

Table 5

Strain Data for Stem at

el 11.0 ft

Date	Measured Compressive Strain, in./in.	Measured Tension Strain, in./in.
9-22-83	-82	38
12-13-83	-68	73
3-16-85	-47	104
6-11-85	NA	73
9-19-85	NA	60
1-29-86	NA	94

Note: dc = 6.8 in. = distance to comp. steel
dt = 81.9 in. = distance to tension steel
Ac = 4.05 sq in./ft of width = area of compressive steel
At = 5.08 sq in./ft of width = area of tension steel

Table 6

Calculated Forces for Base Slab at

16.5 ft from Center of Chamber

	Axial Force, kips		Bending Moment, ft-lb		
Date	Strain Data	CUFRAM Data	Strain Data	CUFRAM Data	Ratio
9-22-83	12.2	-33.3	1.14E6	1.98E6	1.74
12-13-83	17.7	-36.1	1.20E6	1.90E6	1.58
3-16-85	59.1	-31.4	7.70E5	8.43E5	1.09
6-11-85	43.2	-27.9	6.51E5	1.20E6	1.84
9-19-85	9.6	-27.3	1.01E6	1.75E6	1.73
1-29-86	71.8	-34.1	1.06E6	1.23E6	1.16

Note: Positive force is tension; positive moment gives compression on chamber side.

Table 7

Calculated Forces for Base Slab at 42 ft from Center of Chamber

Date	Axial Force, kips		Bending Moment, ft-lb		
	Strain Data	CUFRAM Data	Strain Data	CUFRAM Data	Ratio
9-22-83	42.3	-35.1	1.71E5	5.46E5	3.19
12-13-83	52.6	-37.6	2.11E5	4.86E5	2.30
3-16-85	53.4	-38.8	1.38E5	5.95E5	4.31
6-11-85	37.2	-29.8	8.33E5	2.75E5	3.30
9-19-85	43.1	-29.8	1.24E5	4.44E5	3.58
1-29-86	78.8	-36.9	2.84E5	7.69E4	2.70

Note: Positive force is tension; positive moment gives compression on chamber side.

Table 8

Calculated Forces for Stem at el 11.0 ft

Date	Axial Force, kips		Bending Moment, ft-lb		
	Strain Data	CUFRAM Data	Strain Data	CUFRAM Data	Ratio
9-22-83	-99.7	-43.7	2.59E5	1.17E5	0.45
12-13-83	-58.9	-43.7	2.20E5	1.28E5	0.58
3-16-85	-22.8	-46.6	1.61E5	2.18E5	1.35
6-11-85	NA*	-43.8	NA	1.54E5	--**
9-19-85	NA	-43.5	NA	1.63E5	--
1-29-86	NA	-42.6	NA	2.04E5	--

Note: Positive force is tension; positive moment gives compression on chamber side.
* Incomplete strain data.
** No strain data.

Table 9

CUFRAM Forces--Worst-Case Analysis

Section	Axial Load kips	Moment ft-kips	Shear kips
A	49.7	761	65
B	10.4	-809	103
C	7.2	51	45
D	-15.8	-167	4
E	-5.0	58	36
F	48.0	210	18
G	44.7	507	38
H	177.0	984	82
I	133.0	1,590	114
J	138.0	-508	7

Note: See Figure 36 for section locations. Positive axial force is compressive. Positive moment produces compression on the top of the member or on the side toward the chamber center line.

Table 10

Strains and Stresses Based on CUFRAM Analysis--Worst-Case Load

	Calculated Strains, microstrain			Calculated Stresses, ksi	
Section	Tension Steel	Compression Steel	Extreme Fiber Concrete	Tension Steel	Extreme Fiber Concrete
A	625	-197	-267	18.1	0.834
B	750	-143	-205	21.8	0.640
C	*	*	*	*	*
D	365	-25	-104	10.6	0.325
E	*	*	*	*	*
F	205	-81	-139	5.9	0.434
G	265	-88	-126	7.7	0.393
H	350	-265	-317	10.2	0.990
I	625	-159	-265	18.1	0.827
J	20	-70	-72	0.6	0.225

Note: Allowable steel stress = 20 ksi. Allowable concrete stress = 1.35 ksi.

* No data available.

Table 11

Comparison of CUFRAM Shear Forces With Allowable Shear Forces, kips

Section	Axial Force	Shear Force	Allowable Concrete Shear Force, V*
A	49.7	65.8	71.2
B	10.4	103.0	67.0
C	7.2	45.0	44.8
D	-15.8	3.6	41.2
E	-5.0	36.2	43.5
F	48.0	17.9	46.1
G	44.7	37.9	77.8
H	177.0	82.1	68.5
I	133.0	114.0	98.0
J	138.0	6.7	97.4

* From ACI 318-83 Equation B-2 or B-3.

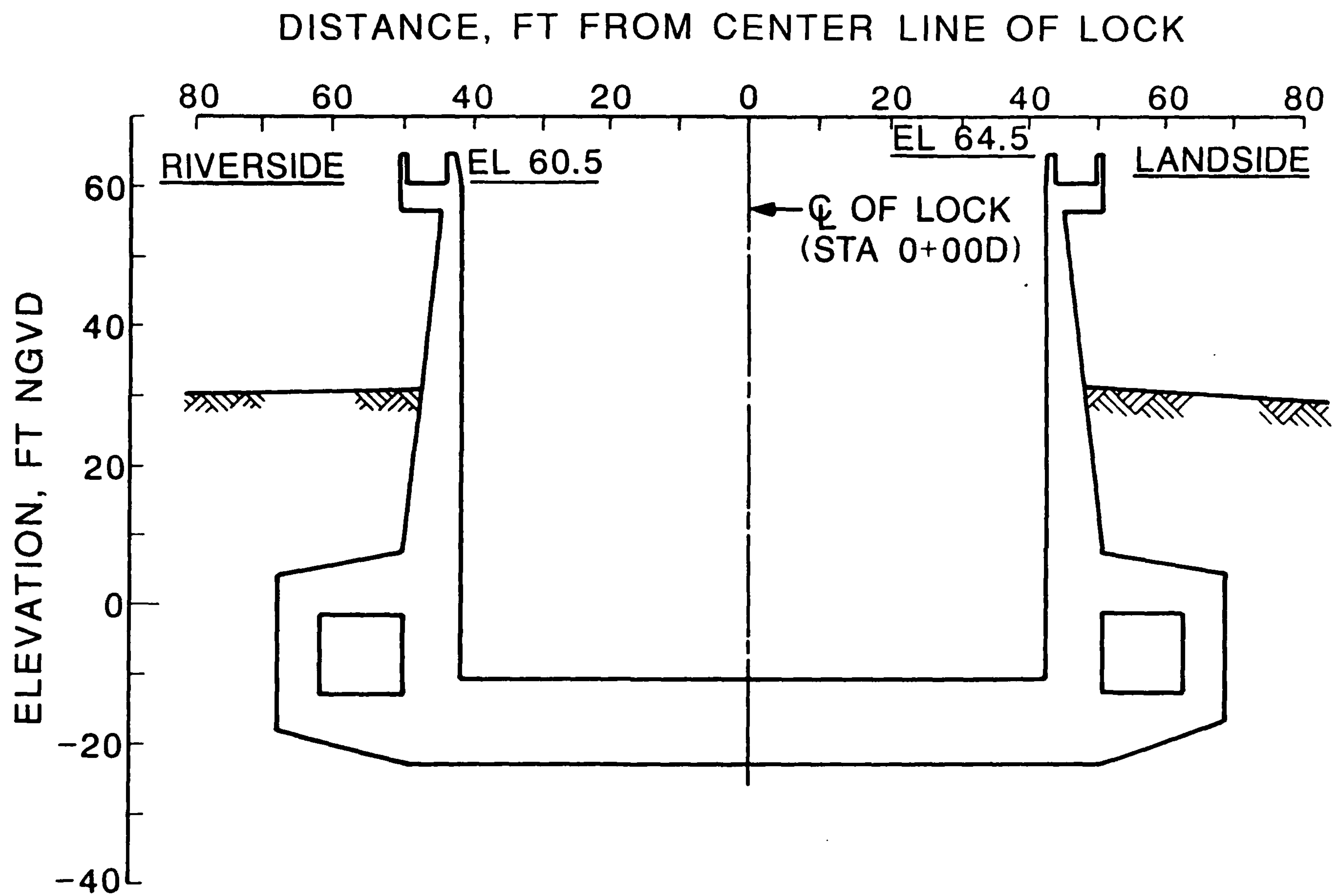


Figure 1. Chamber monolith cross section

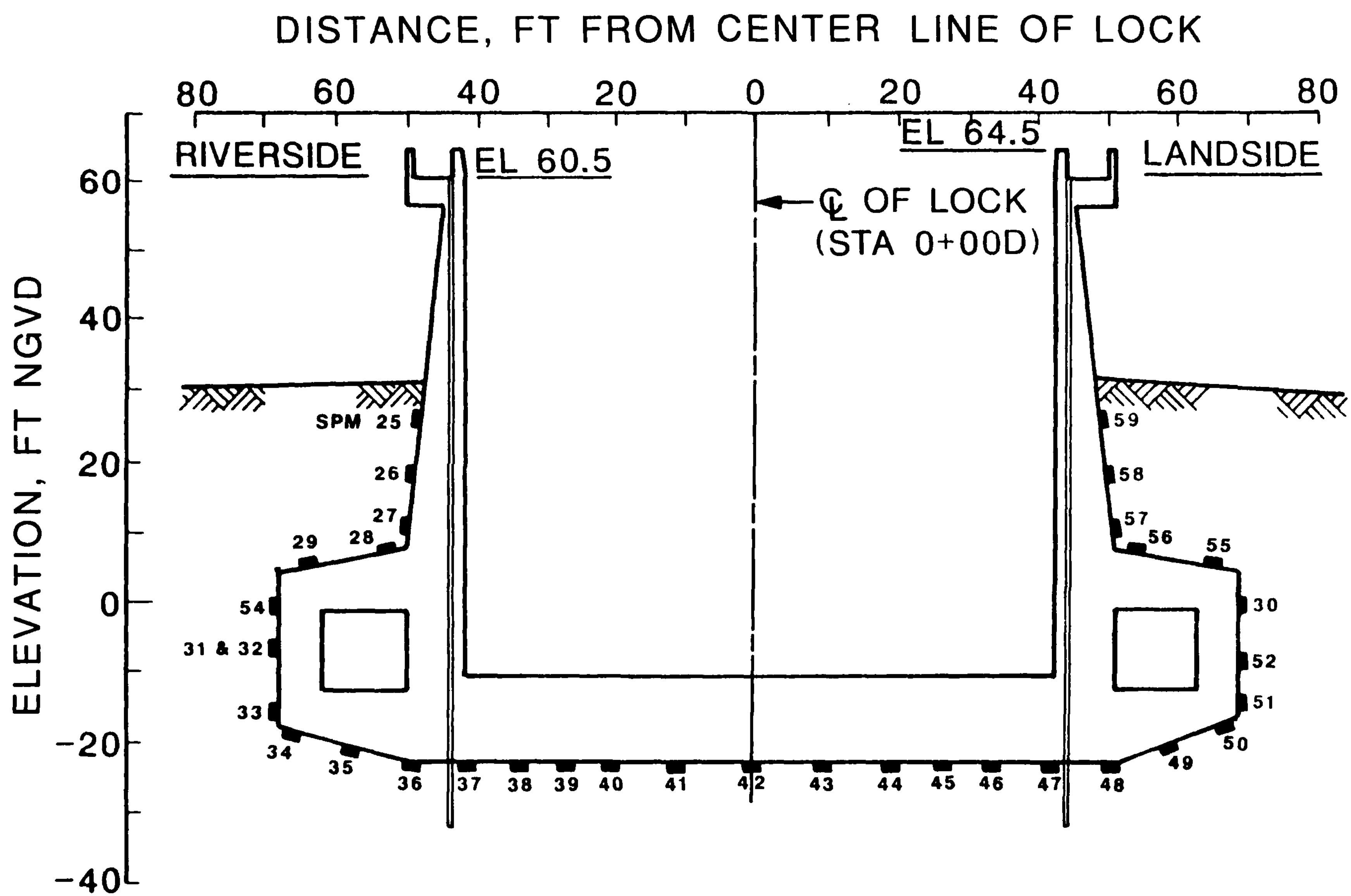
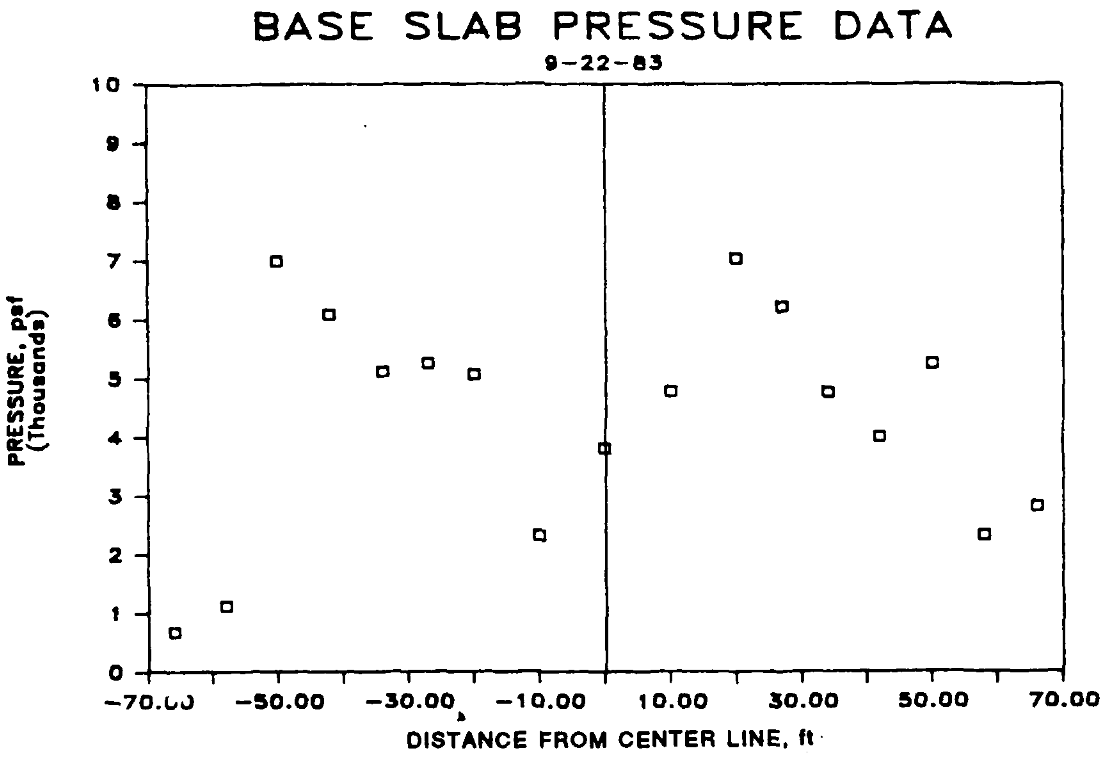


Figure 2. Soil pressure meter locations in Monolith L-10



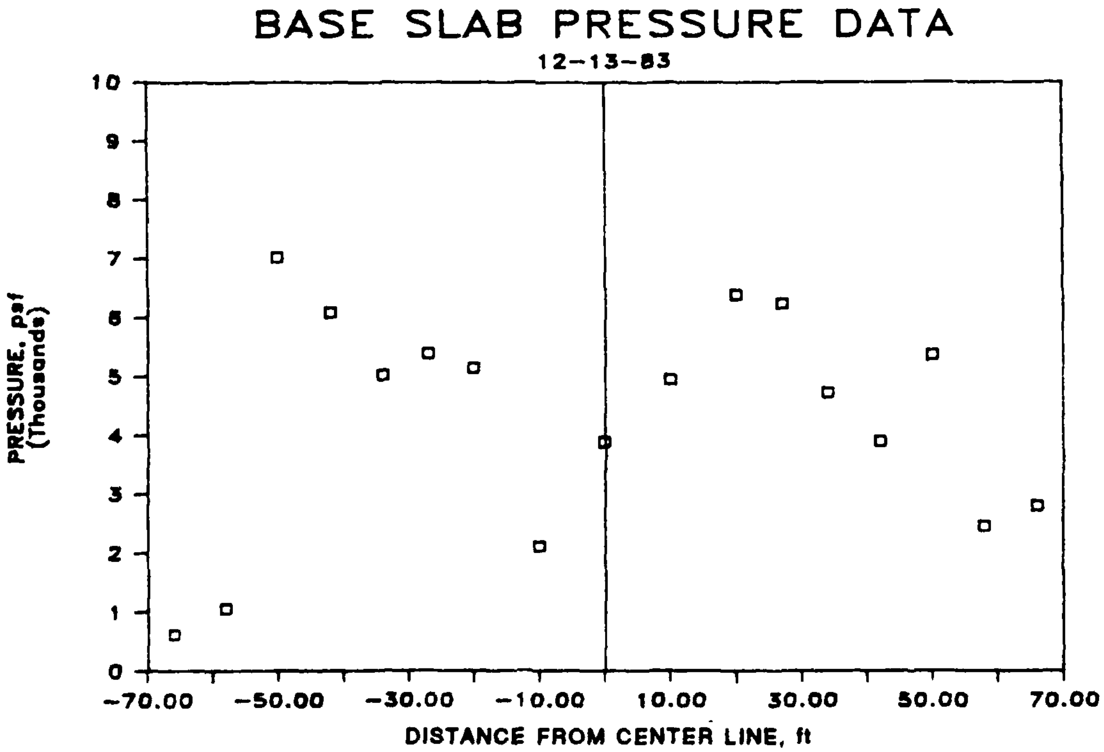
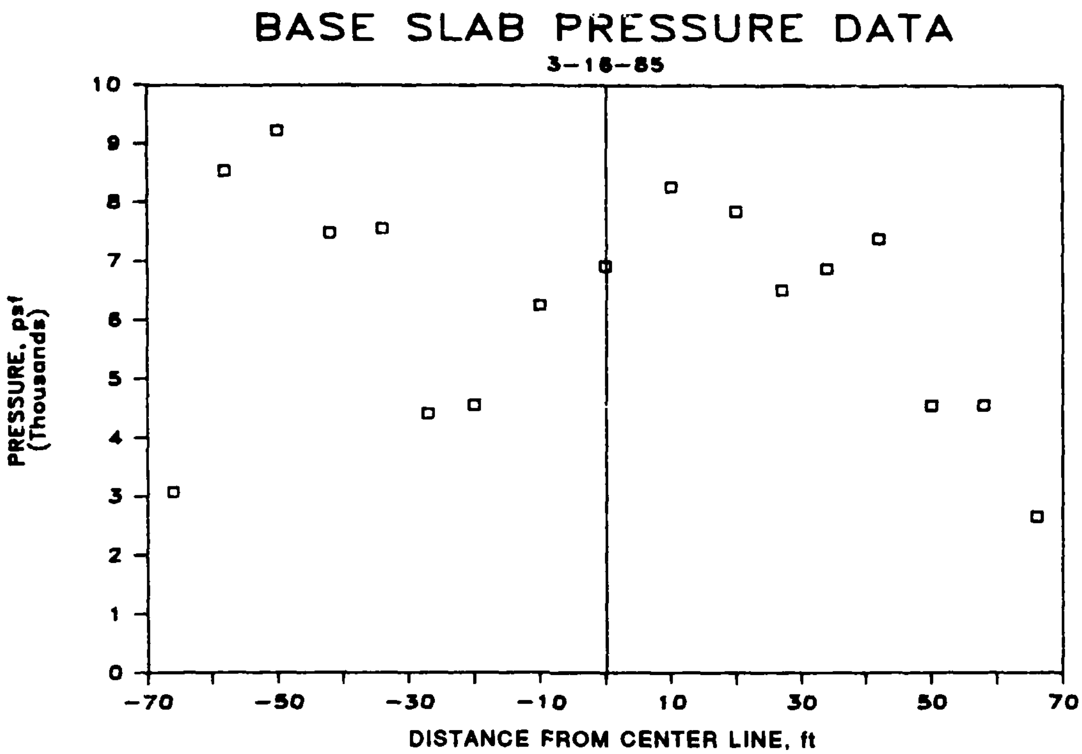


Figure 3. Total pressure distributions for base slab on 9-22-83 and 12-13-83



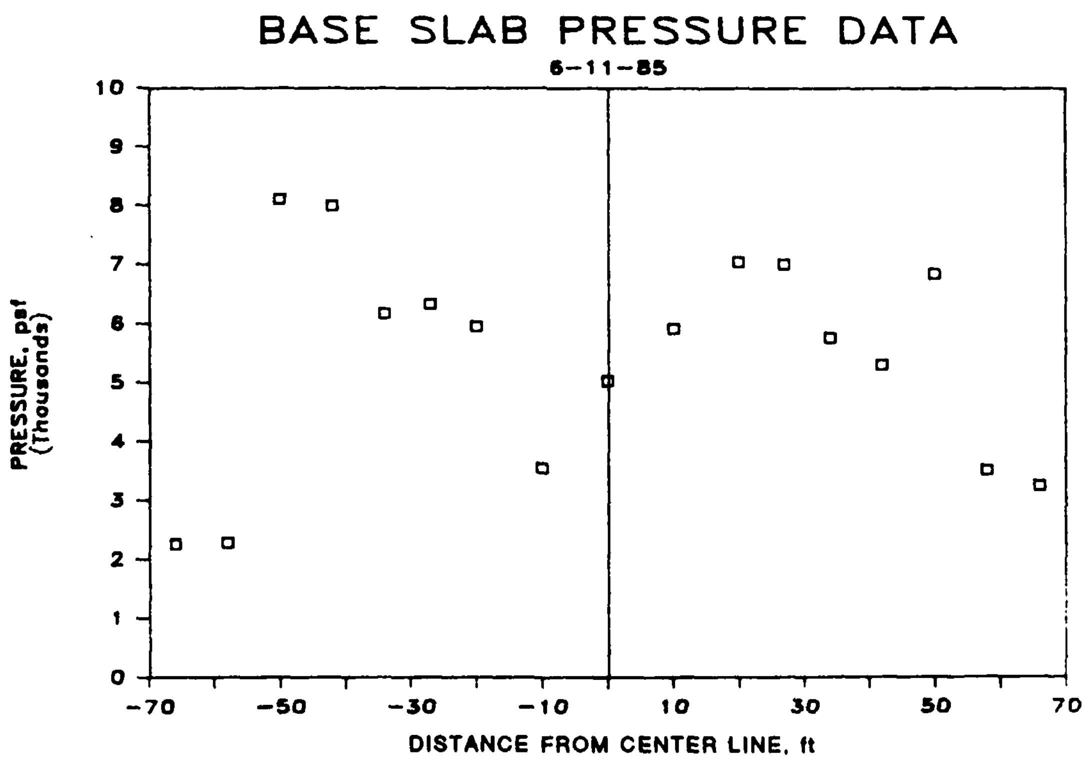
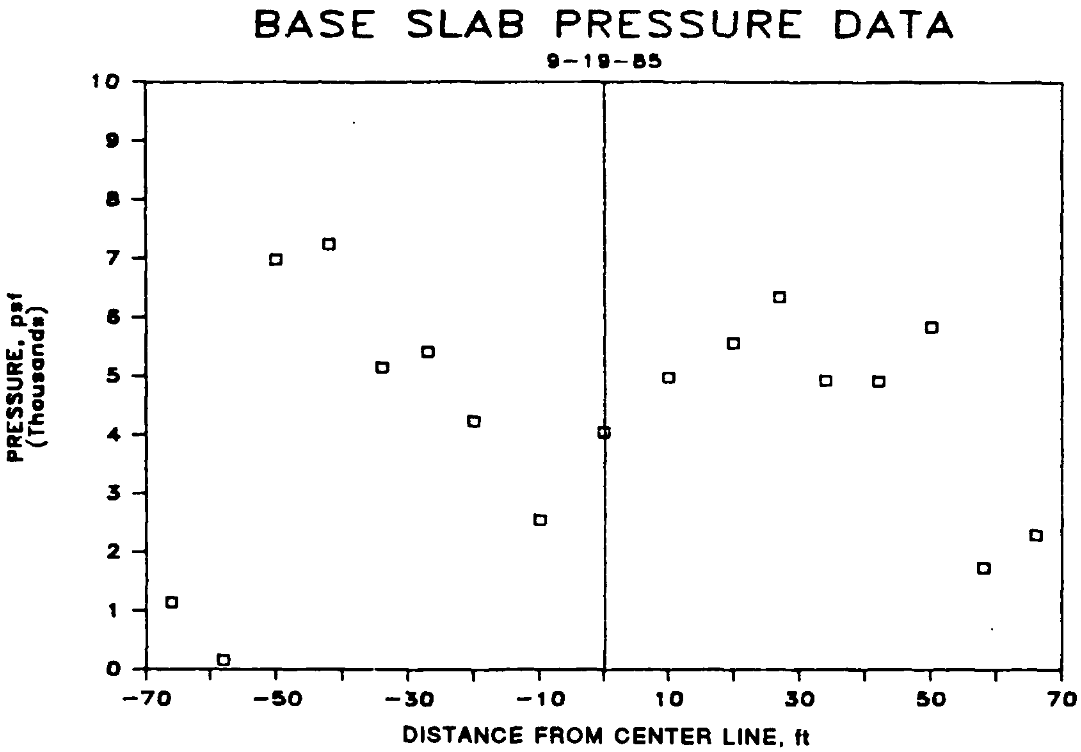


Figure 4. Total pressure distributions for base slab on 3-16-85 and 6-11-85



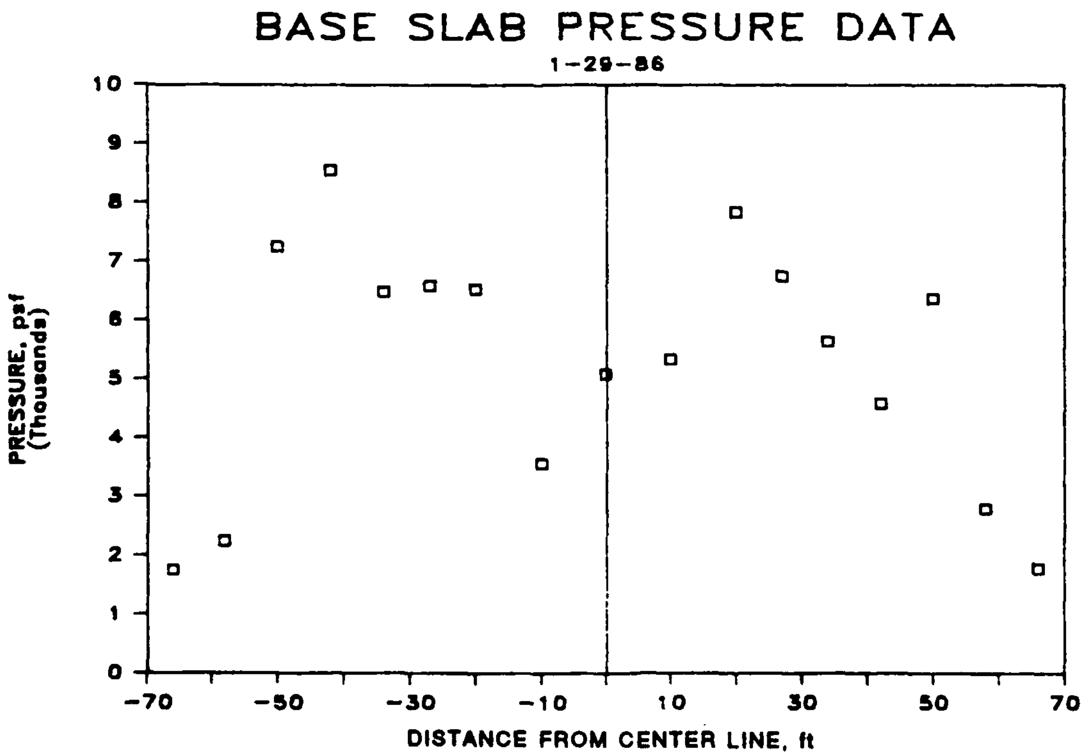
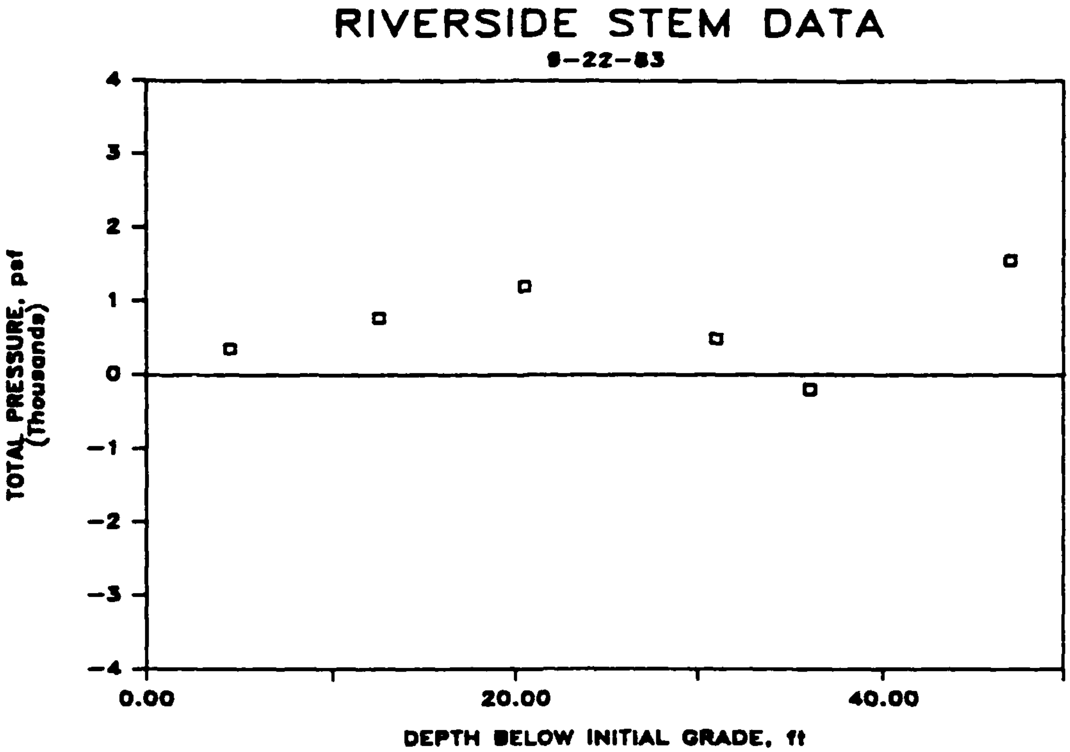


Figure 5. Total pressure distributions for base slab on 9-19-85 and 1-29-86



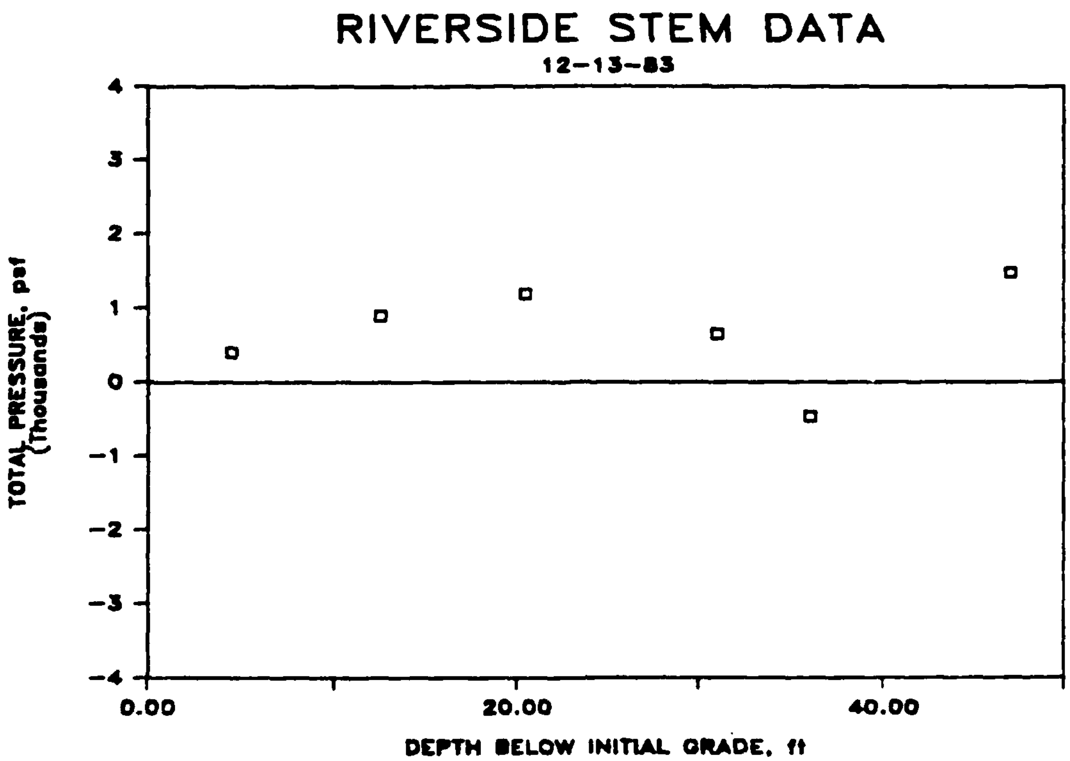
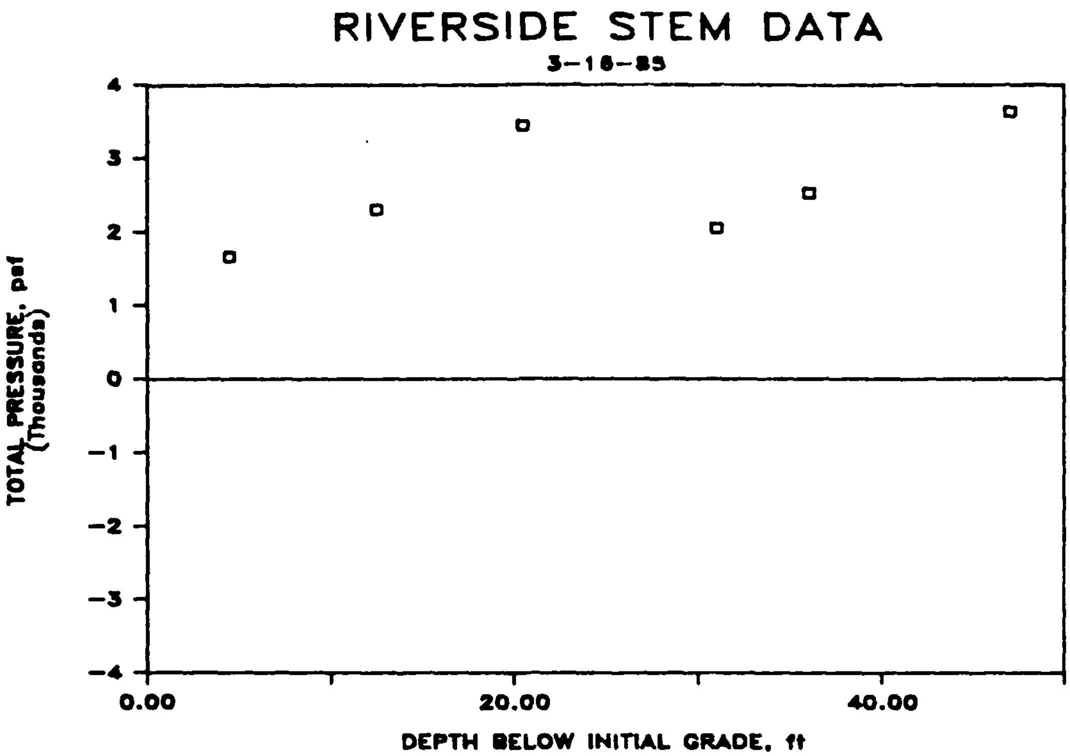


Figure 6. Total pressure distributions for riverside wall on 9-22-83 and 12-13-83



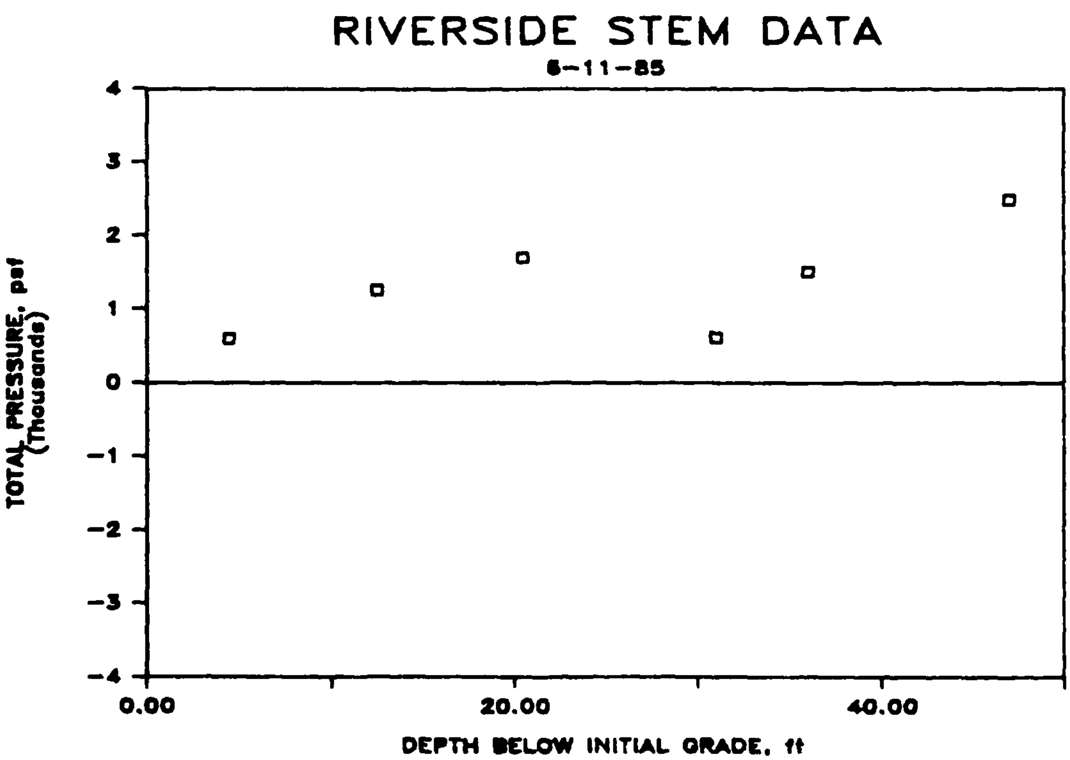
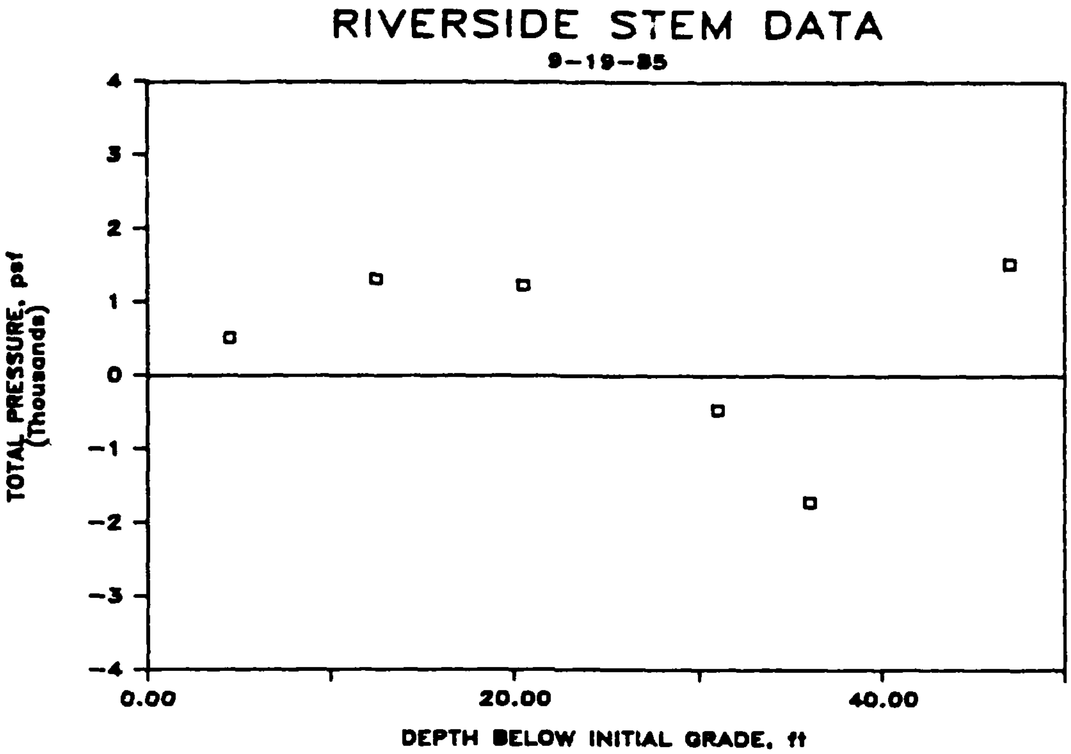


Figure 7. Total pressure distributions for riverside wall on 3-16-85 and 6-11-85



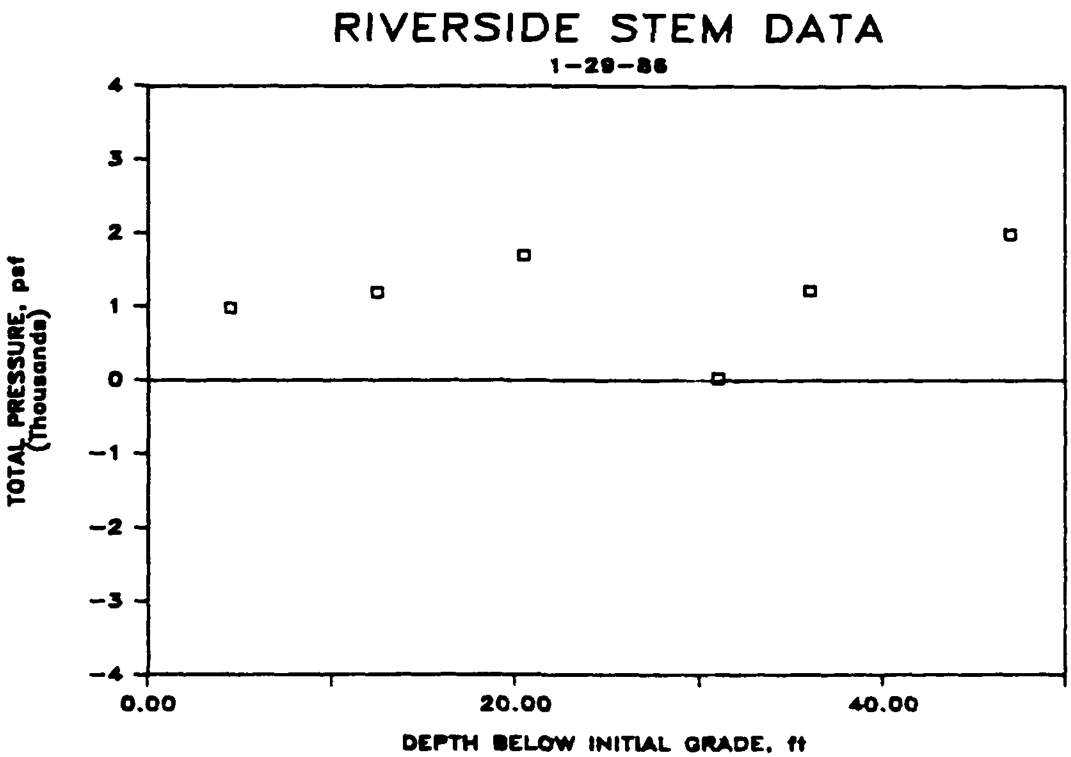
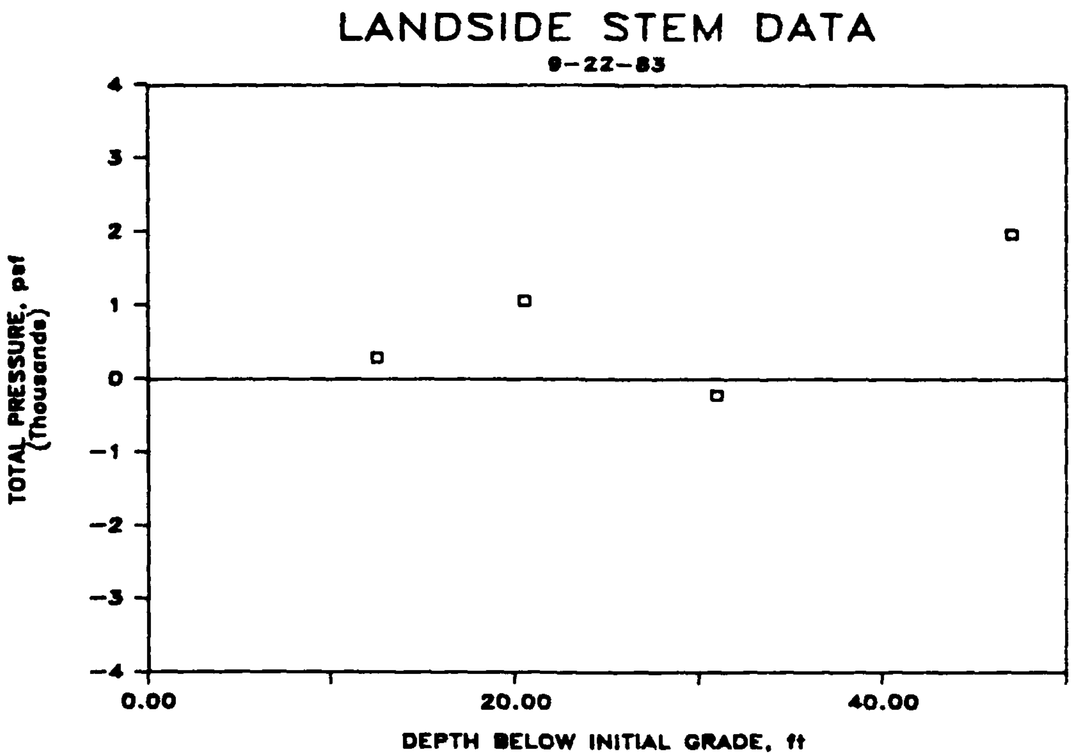


Figure 8. Total pressure distributions for riverside wall on 9-19-85 and 1-29-86



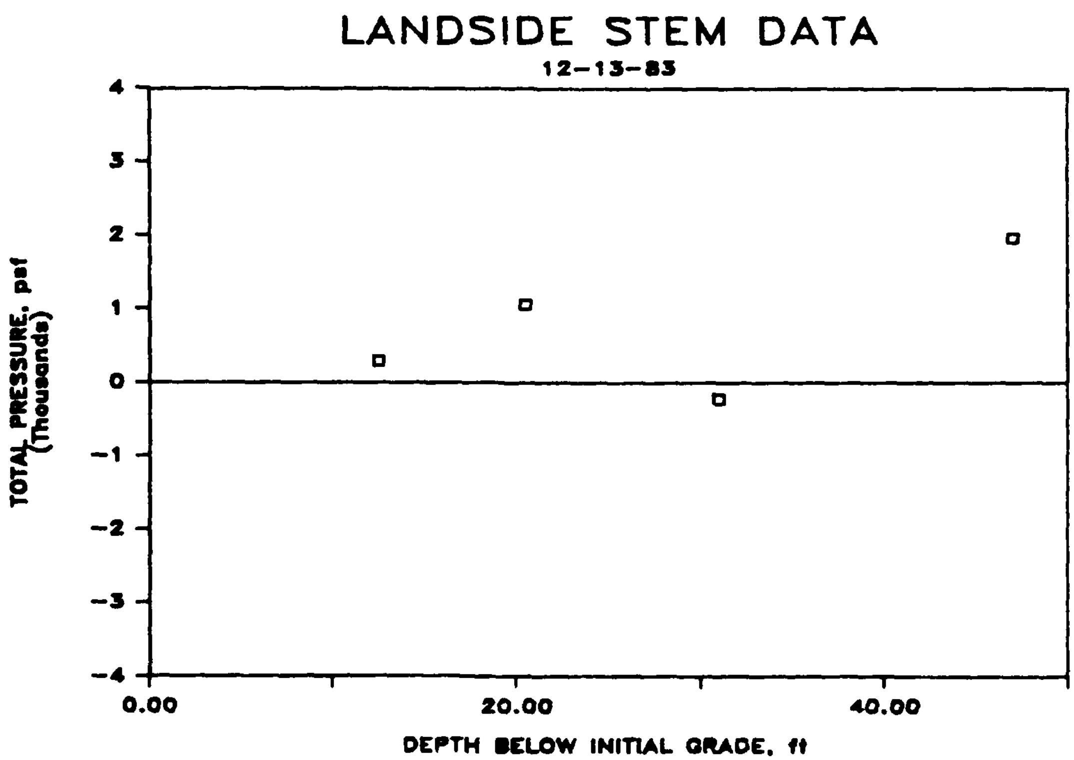
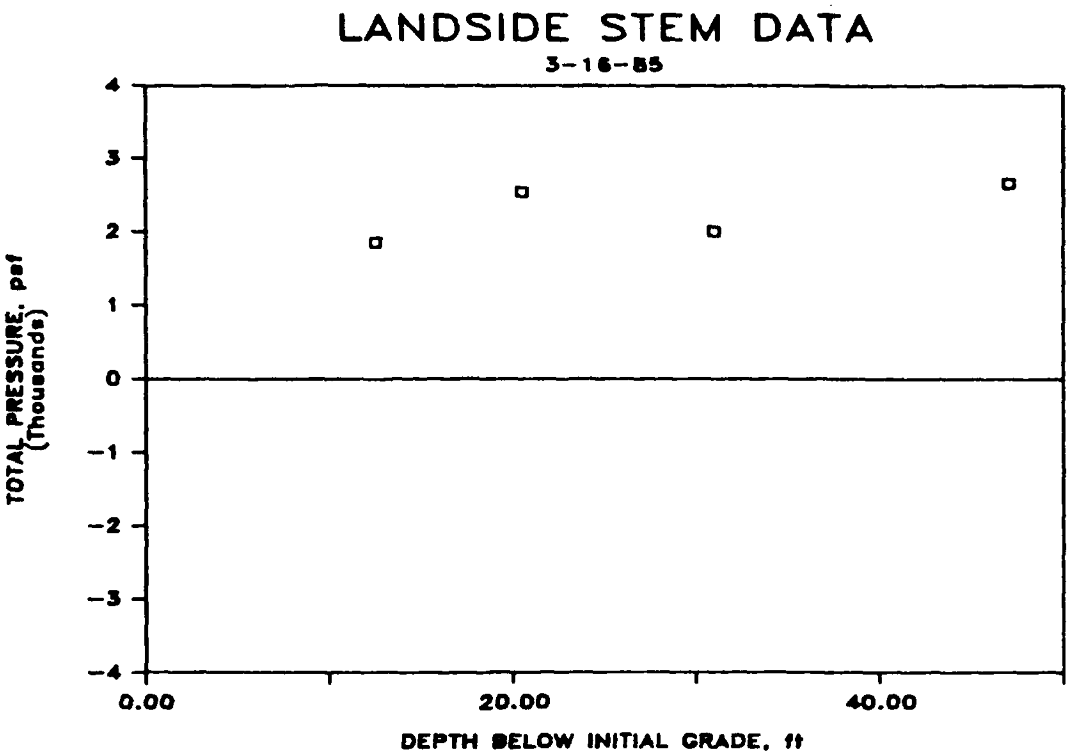


Figure 9. Total pressure distributions for landside wall on 9-22-83 and 12-13-83



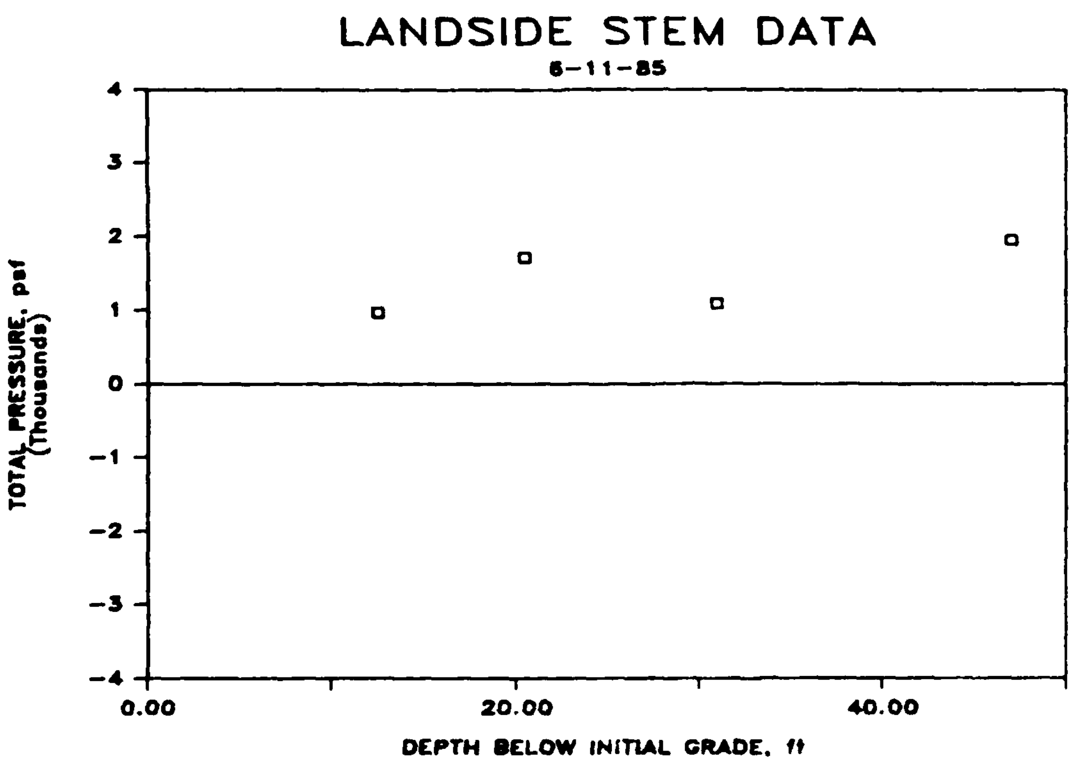
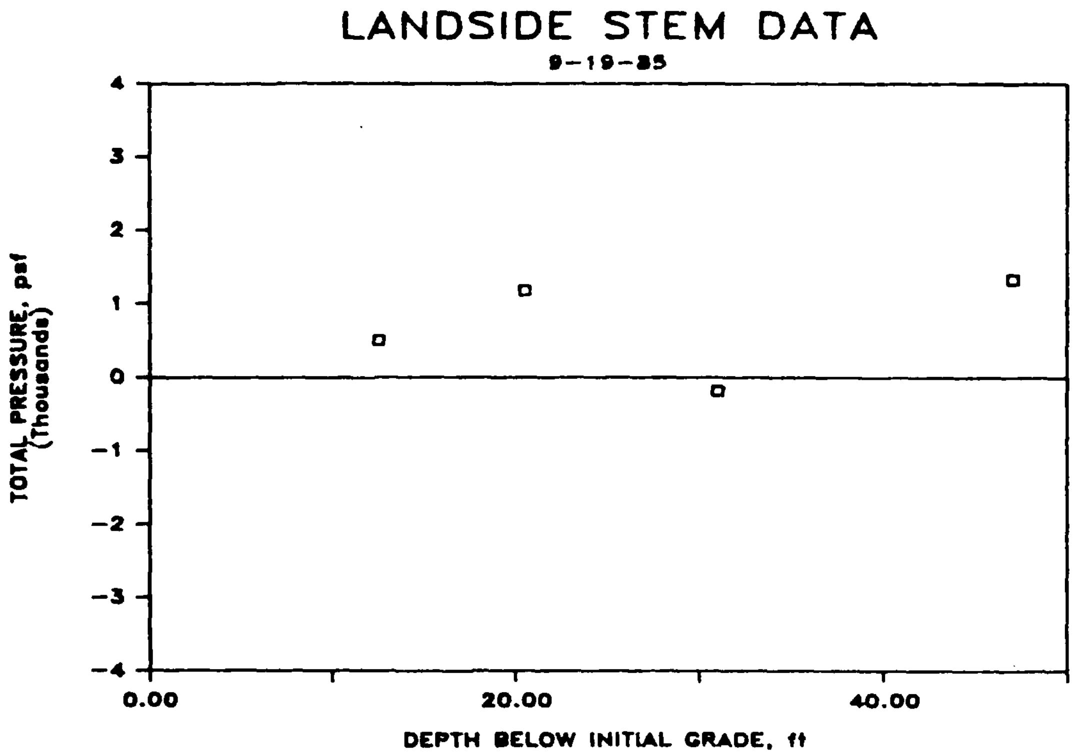


Figure 10. Total pressure distributions for landside wall on 3-16-85 and 6-11-85



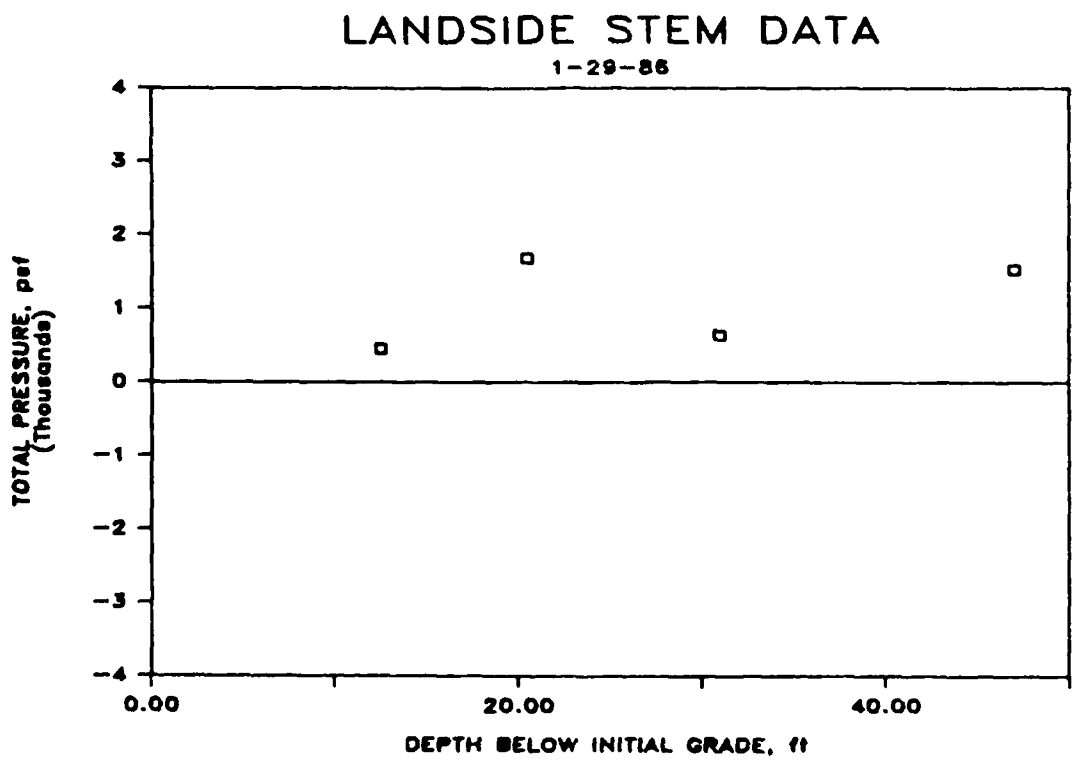


Figure 11. Total pressure distributions for landside wall on 9-19-85 and 1-29-86

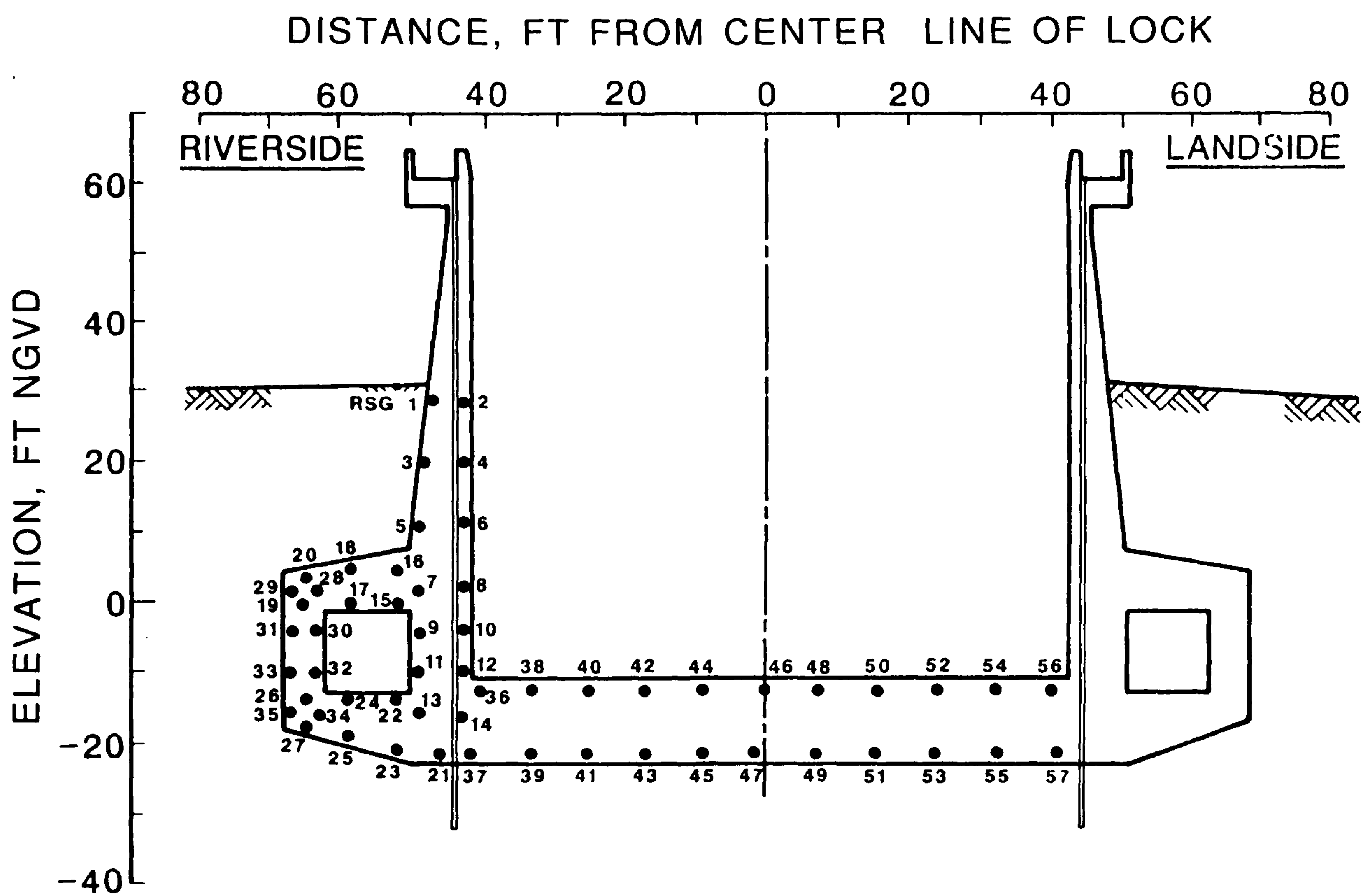


Figure 12. Reinforcing steel strain gage location in Monolith L-10

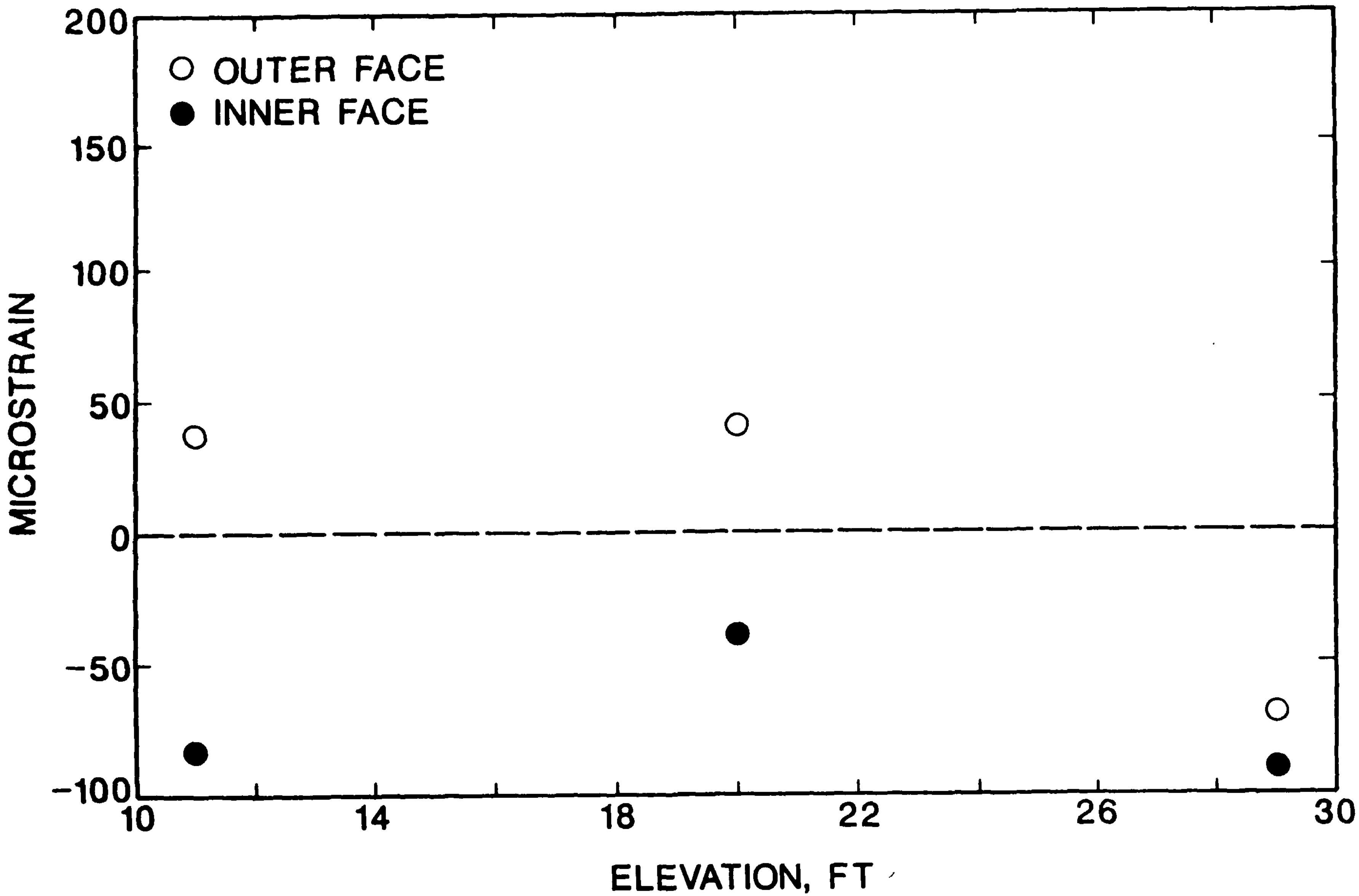


Figure 13. Strain distributions in riverside stem wall on 9-22-83

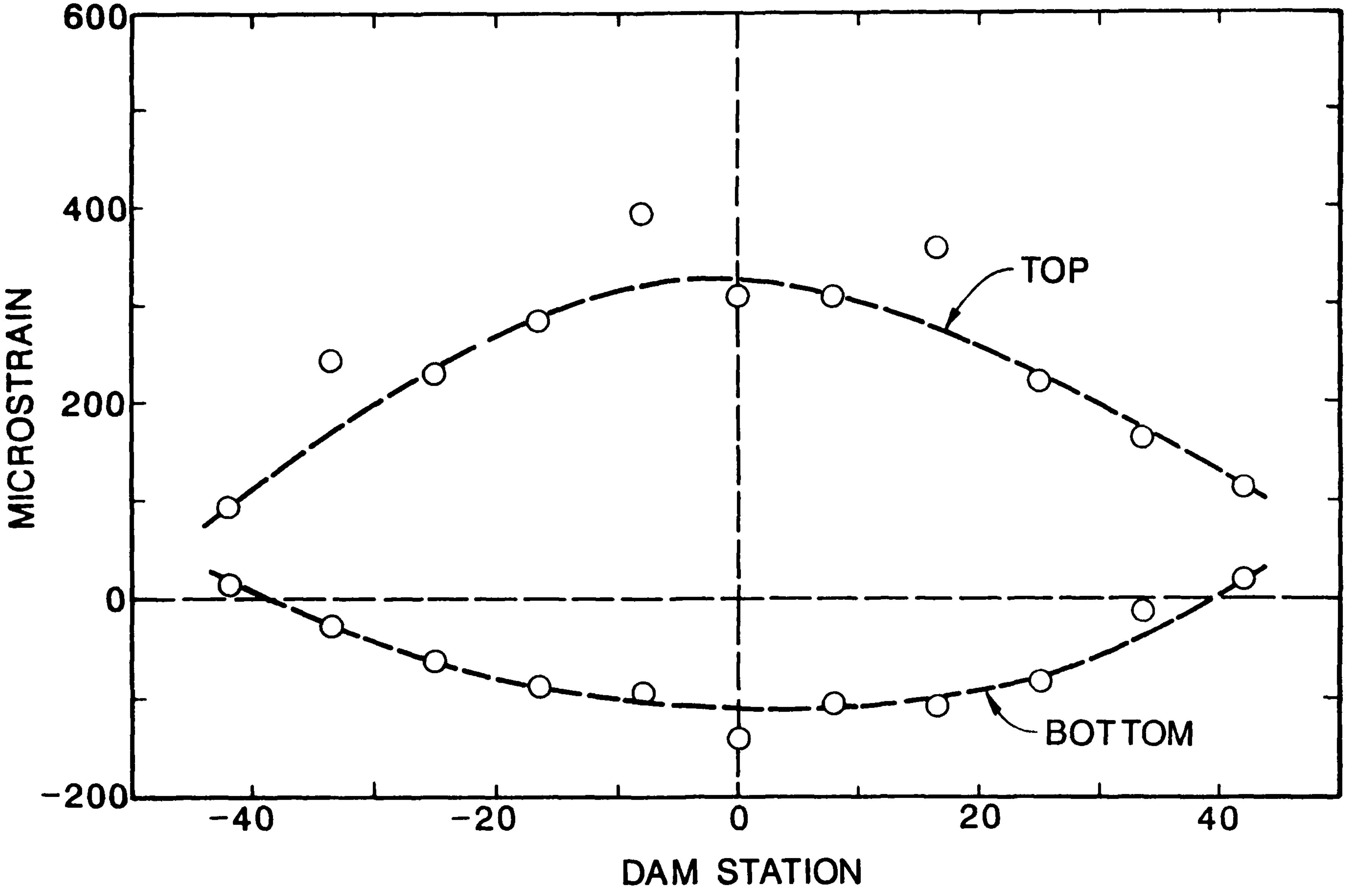


Figure 14. Strain distributions in base slab on 9-22-83

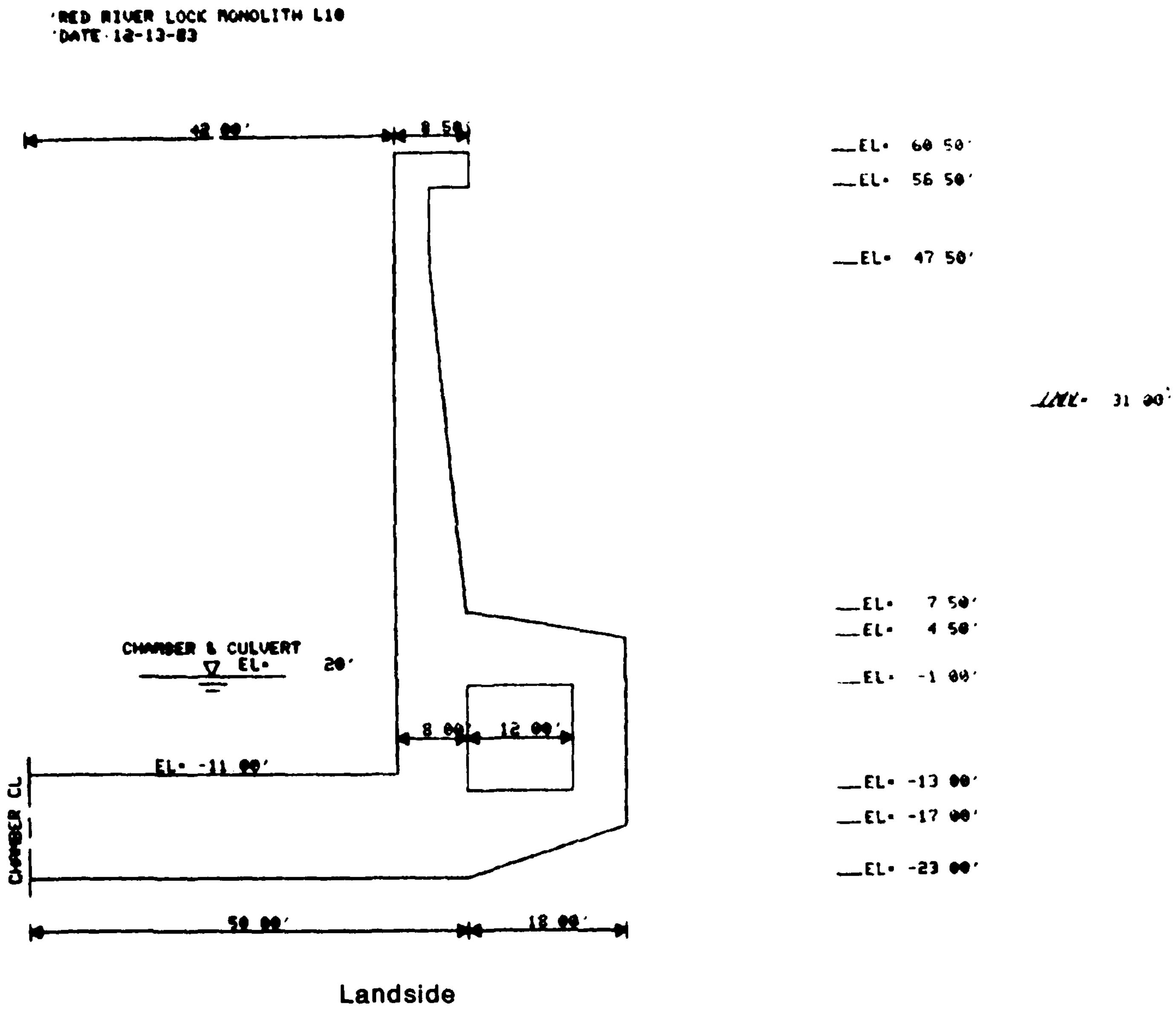


Figure 15. Typical structure definition for CUFRAM analysis

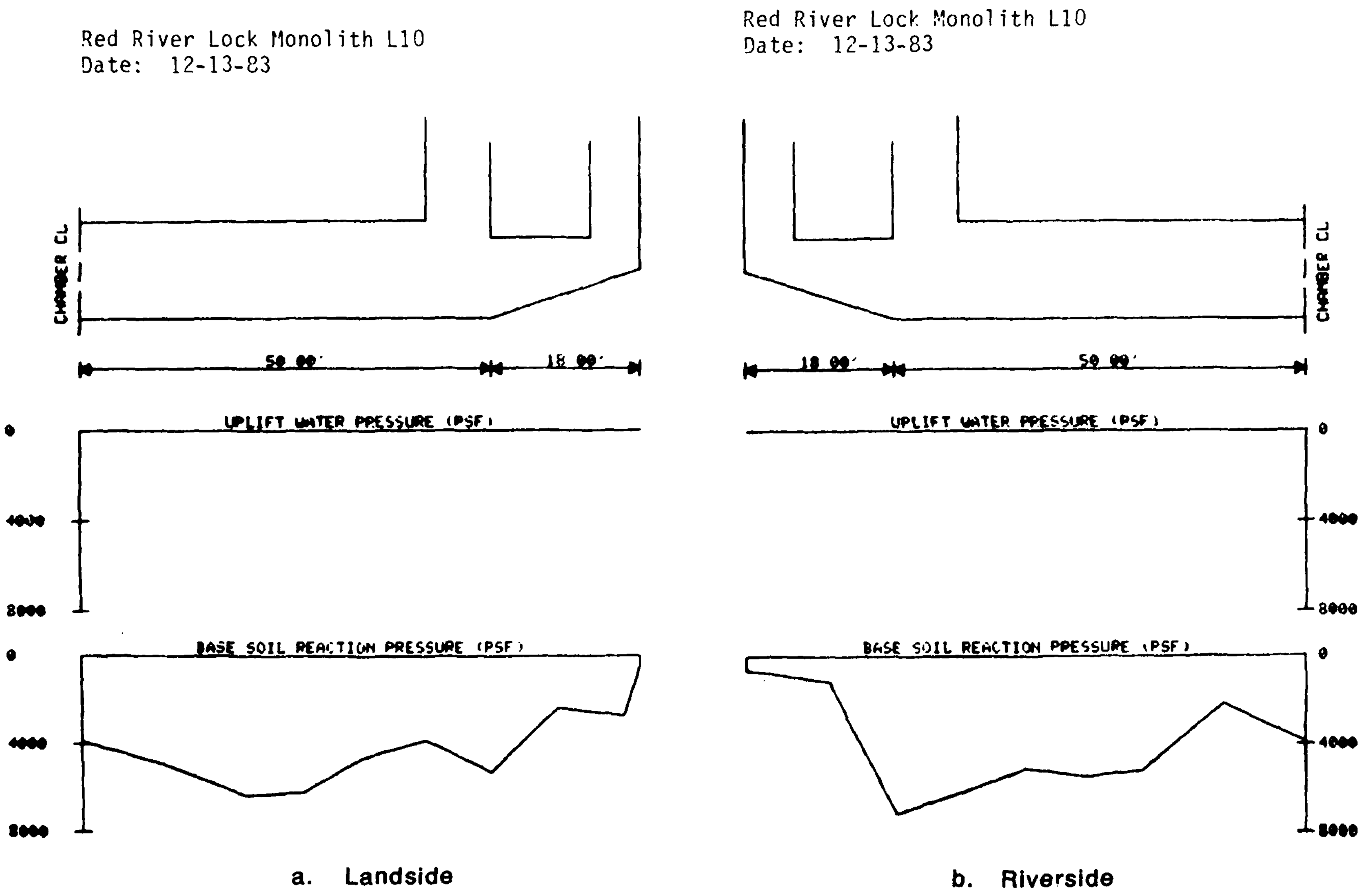
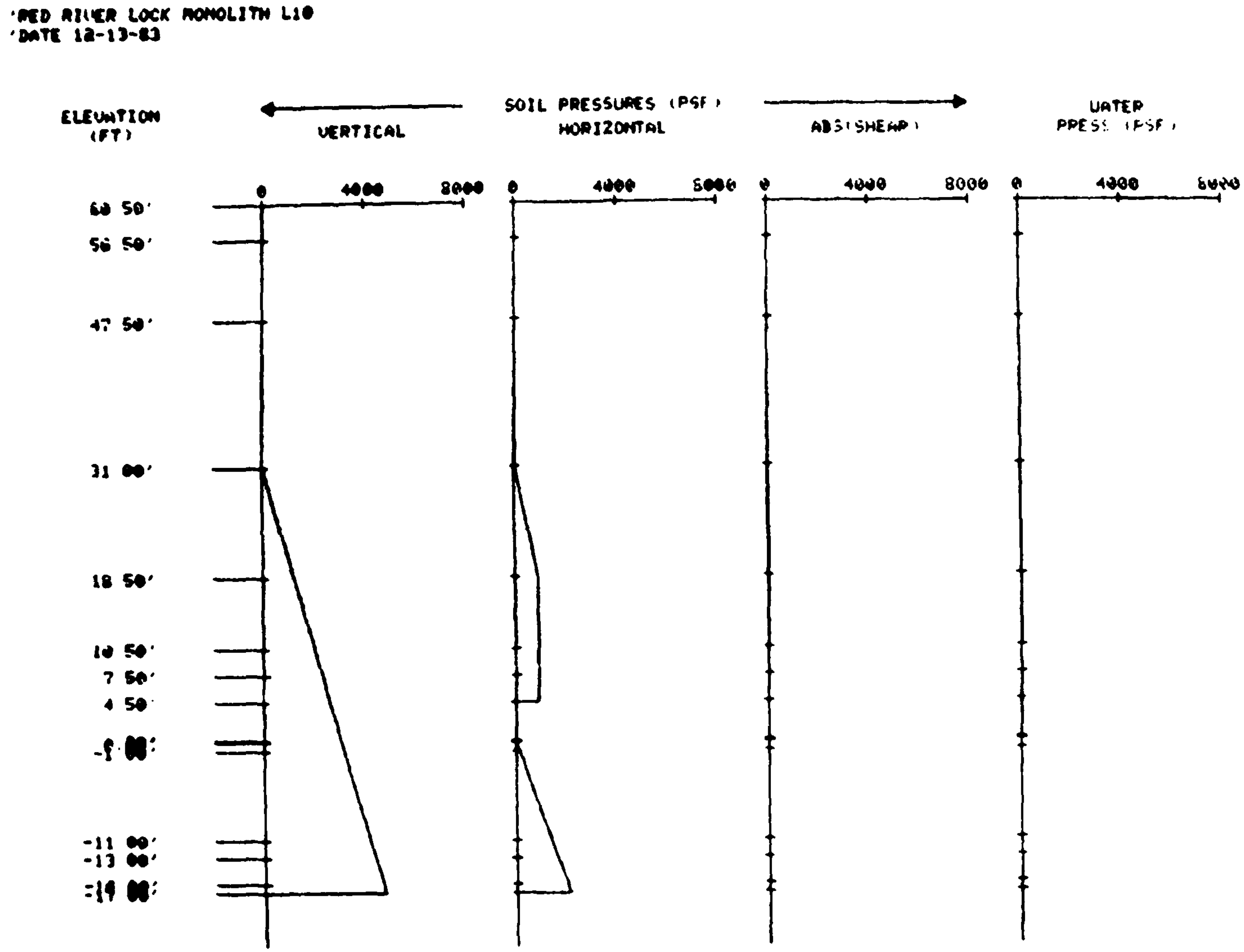


Figure 16. CUFRAM input for base pressure on 12-13-83



Landside

Figure 17. CUFRAM input for landside wall pressure on 12-13-83

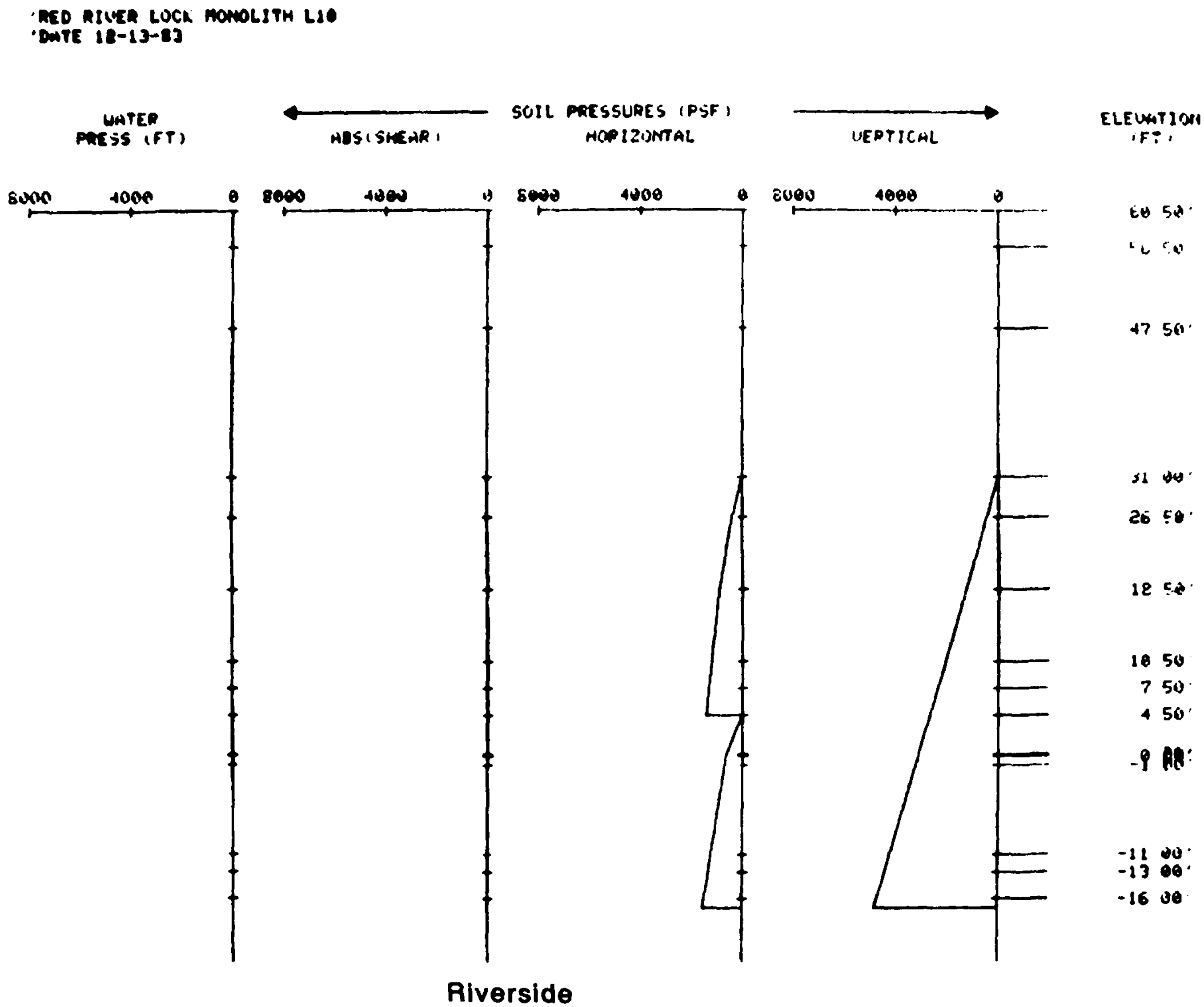
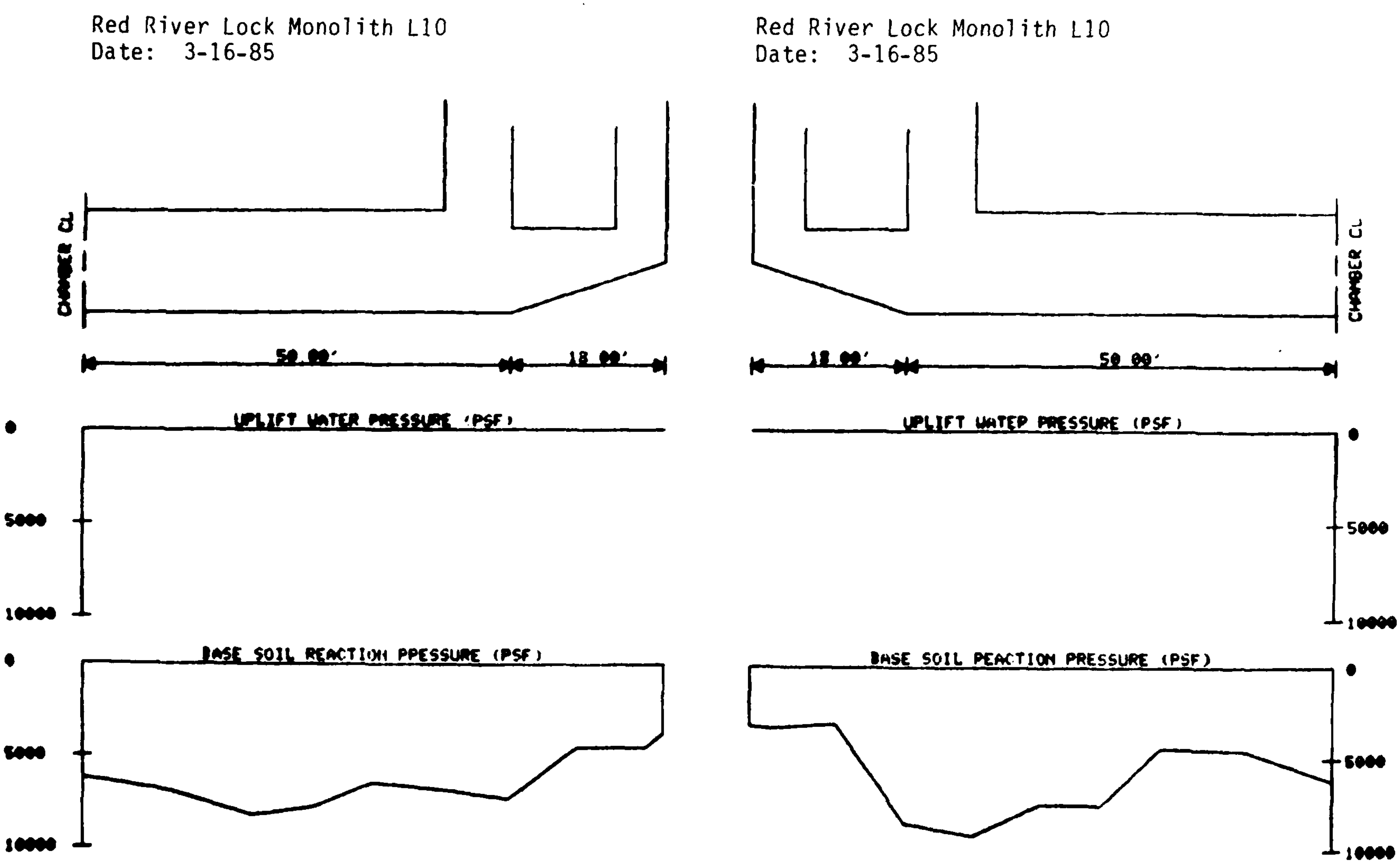


Figure 18. CUFRAM input for riverside wall pressure on 12-13-83



a. Landside

b. Riverside

Figure 19. CUFRAM input for base pressure on 3-16-85

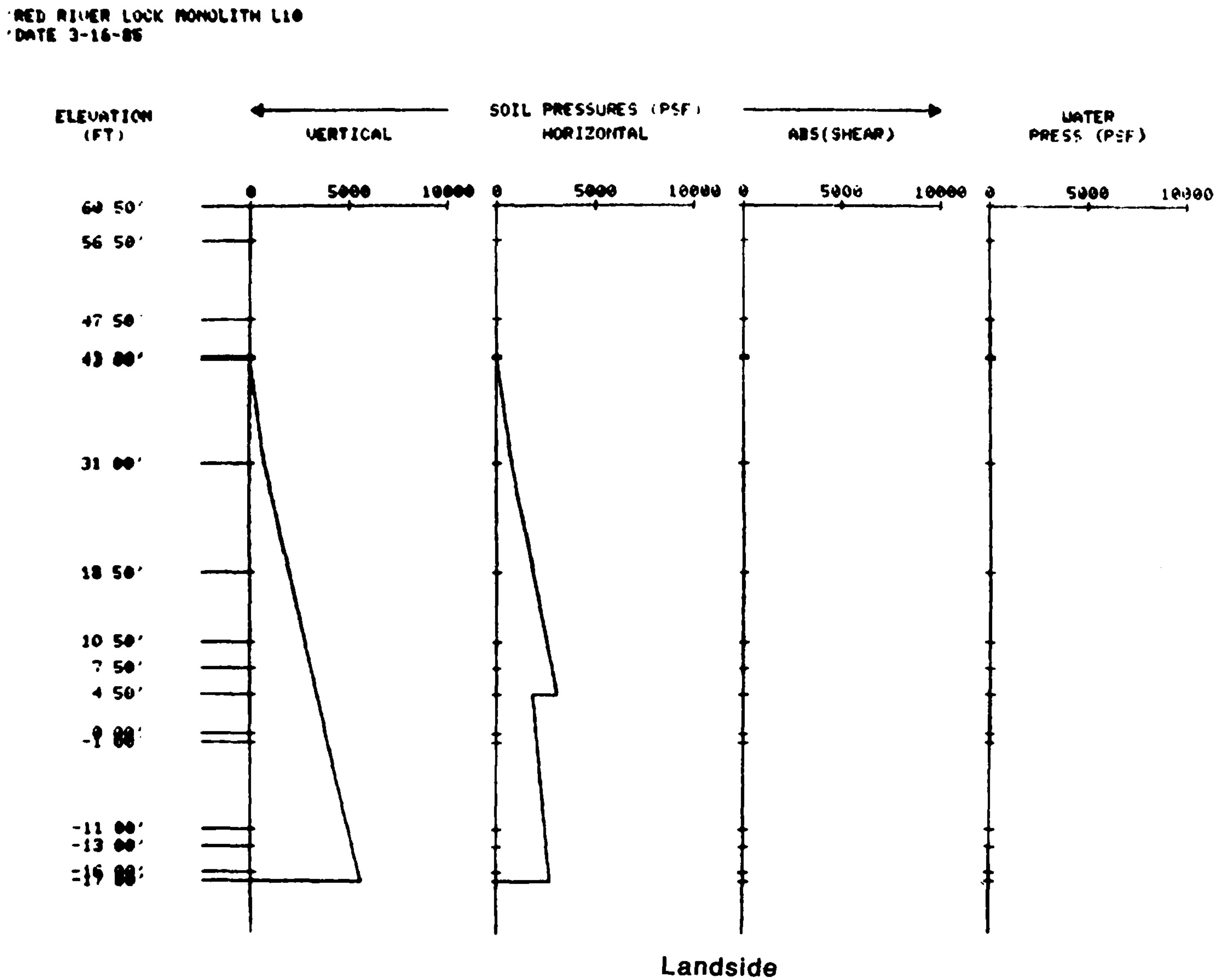
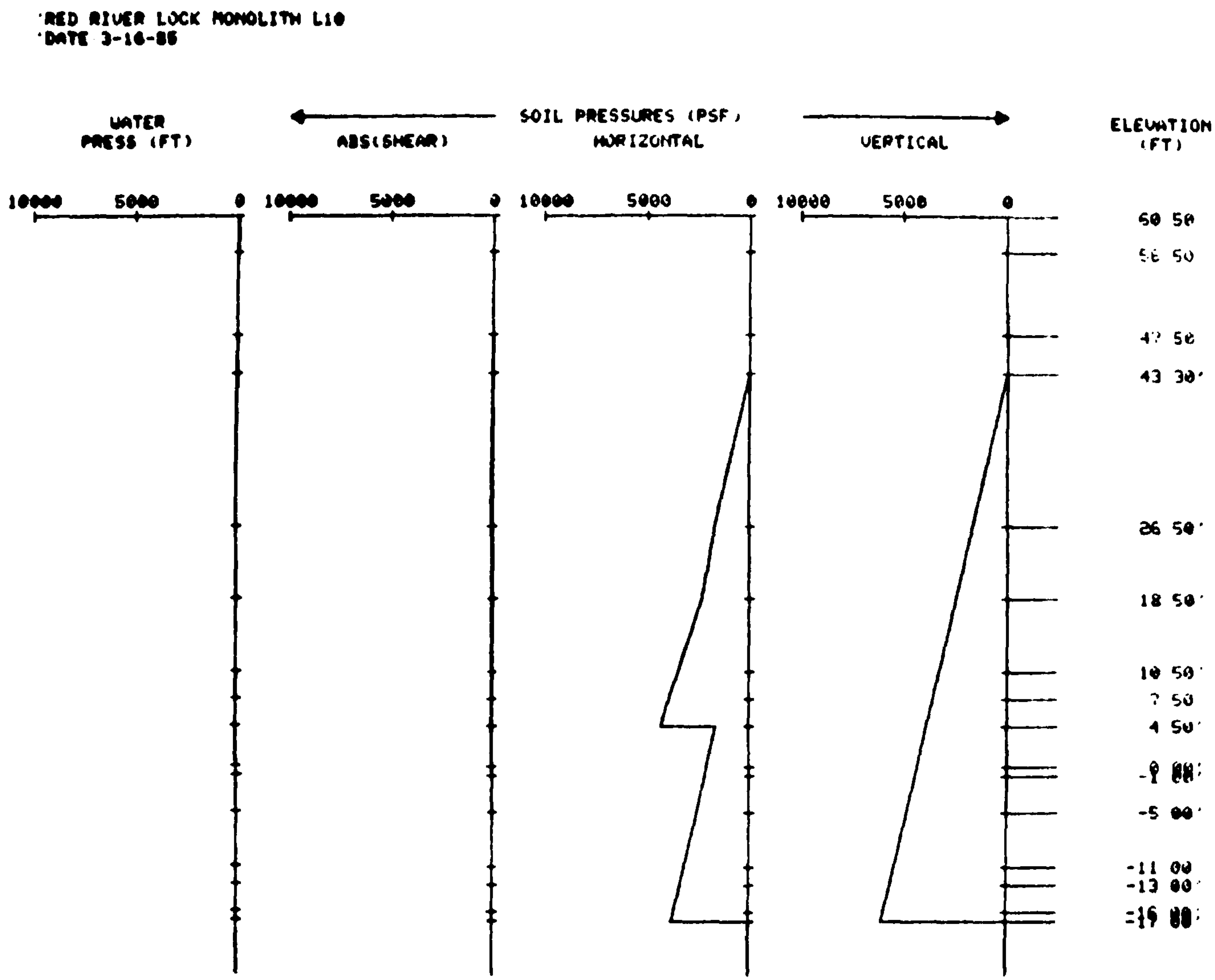


Figure 20. CUFRAM input for landside wall pressure on 3-16-85



Riverside

Figure 21. CUFRAM input for riverside wall pressure on 3-16-85

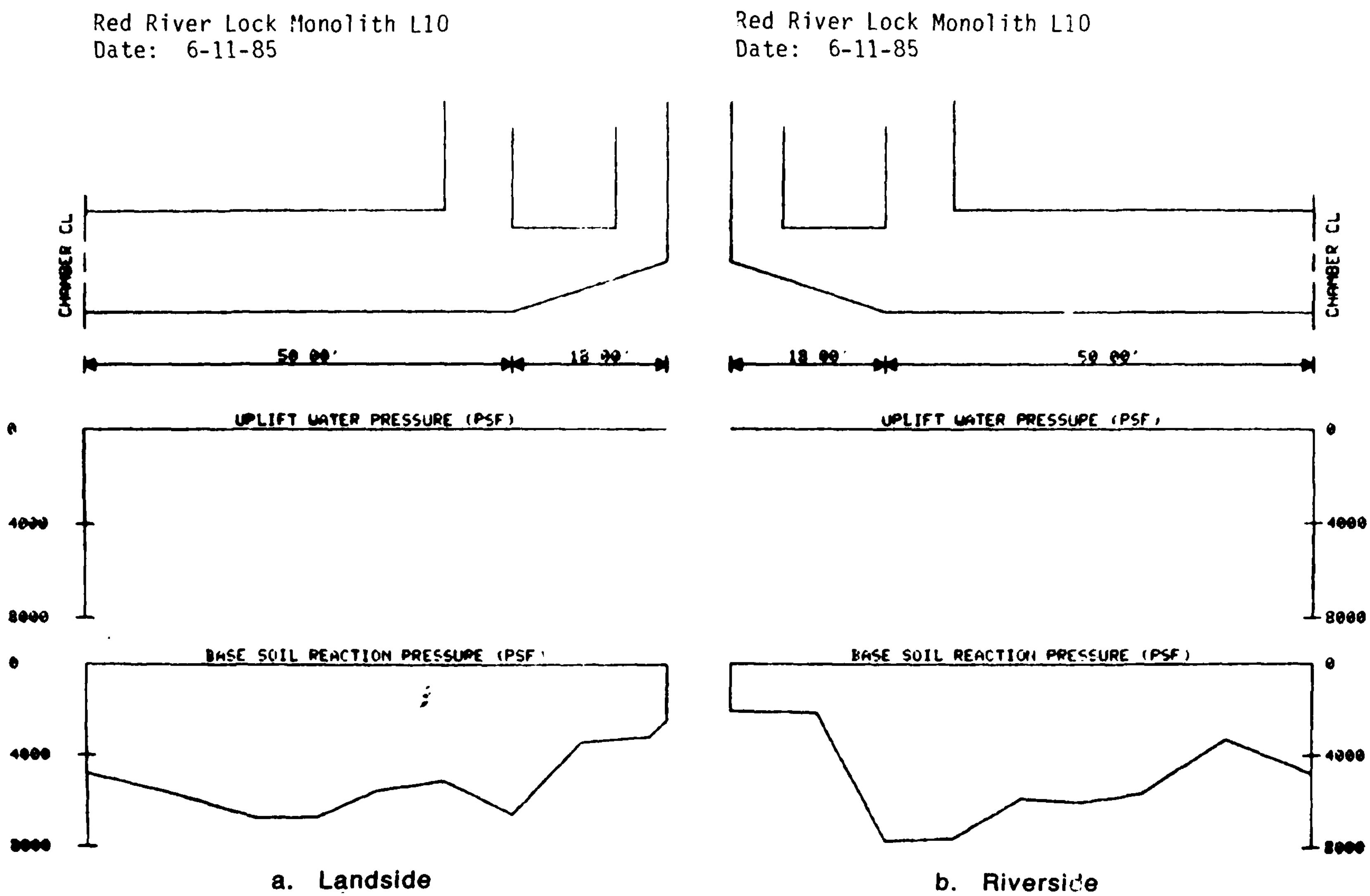


Figure 22. CUFRAM input for base pressure on 6-11-85

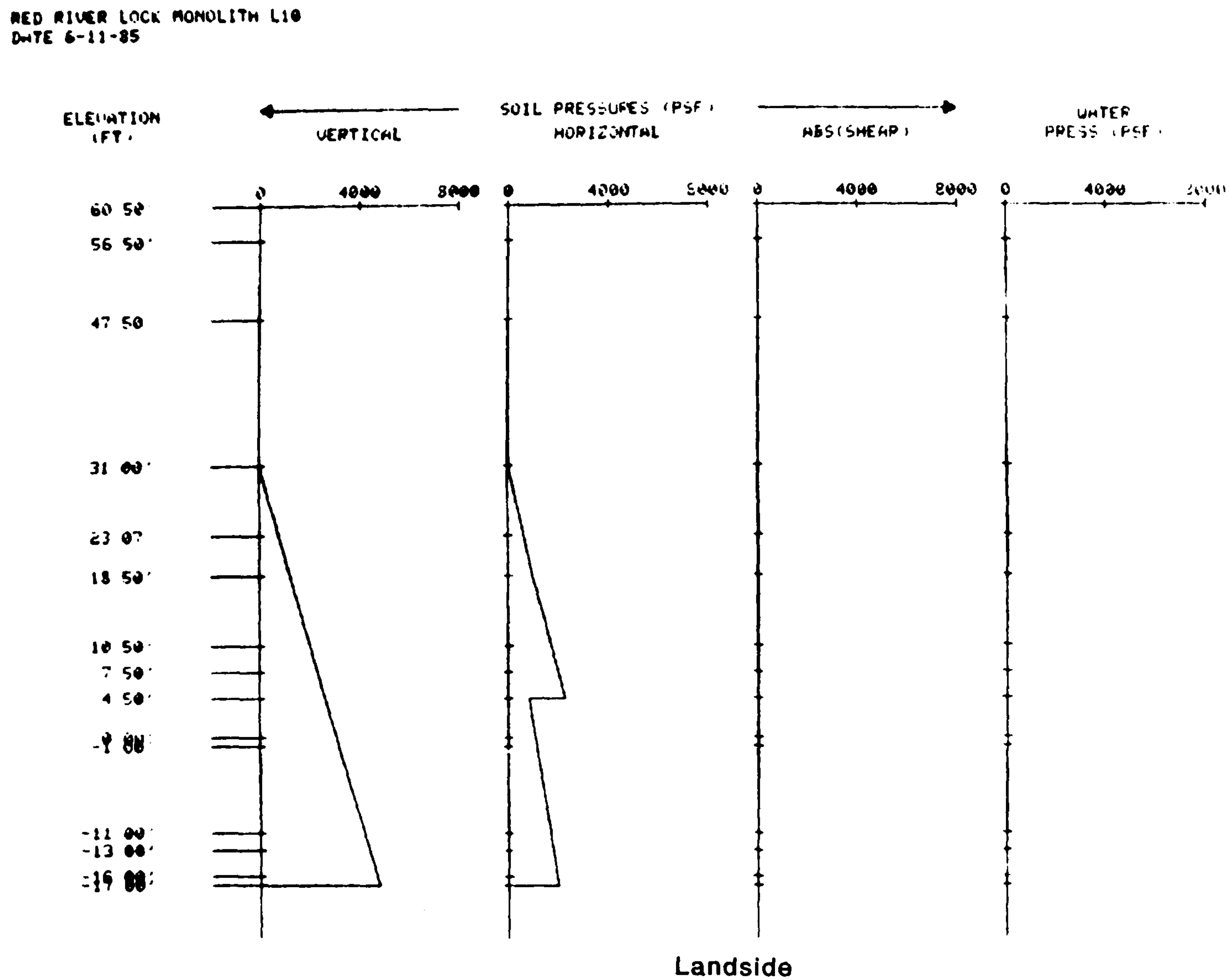
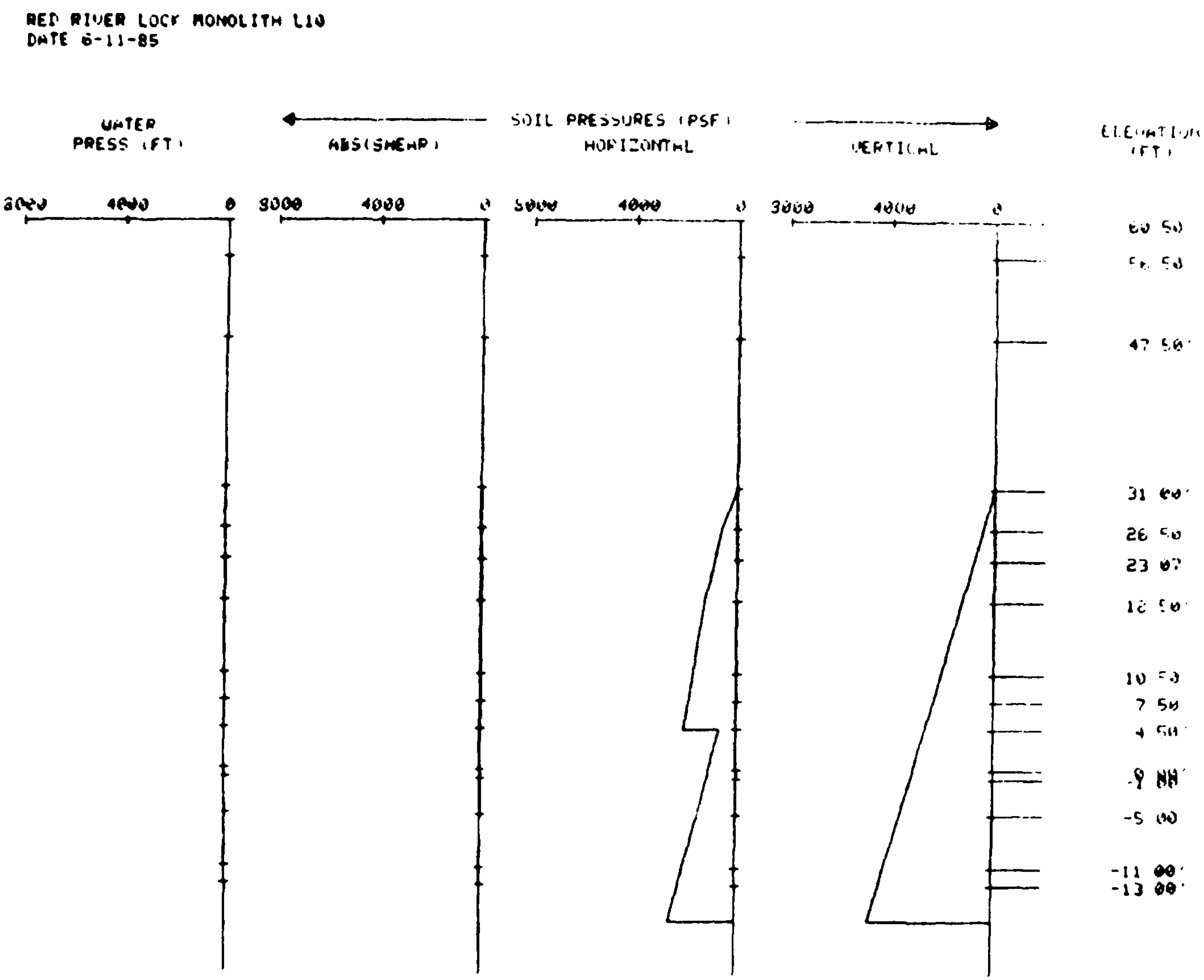
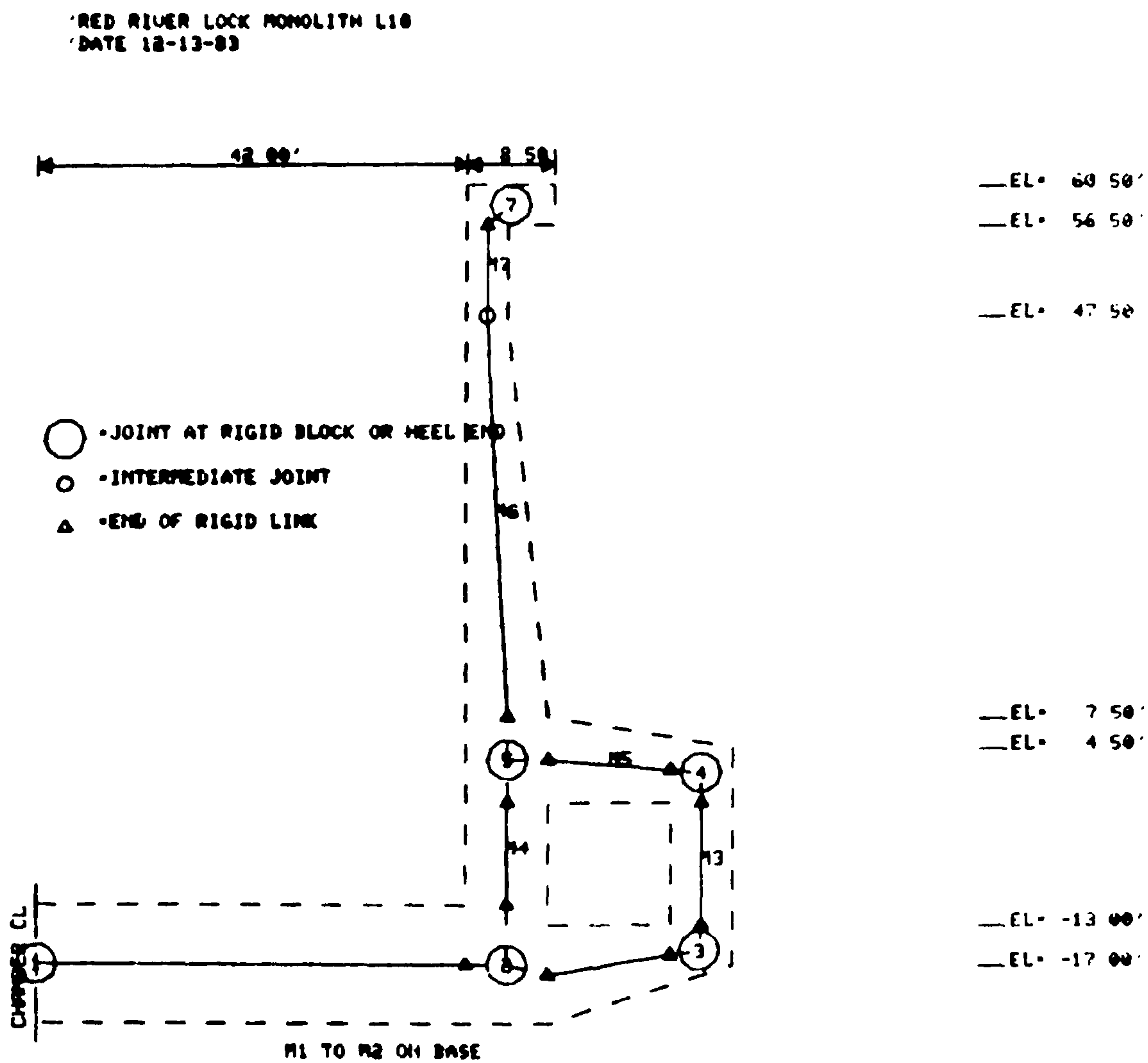


Figure 23. CUFRAM input for landside wall pressure on 6-11-85



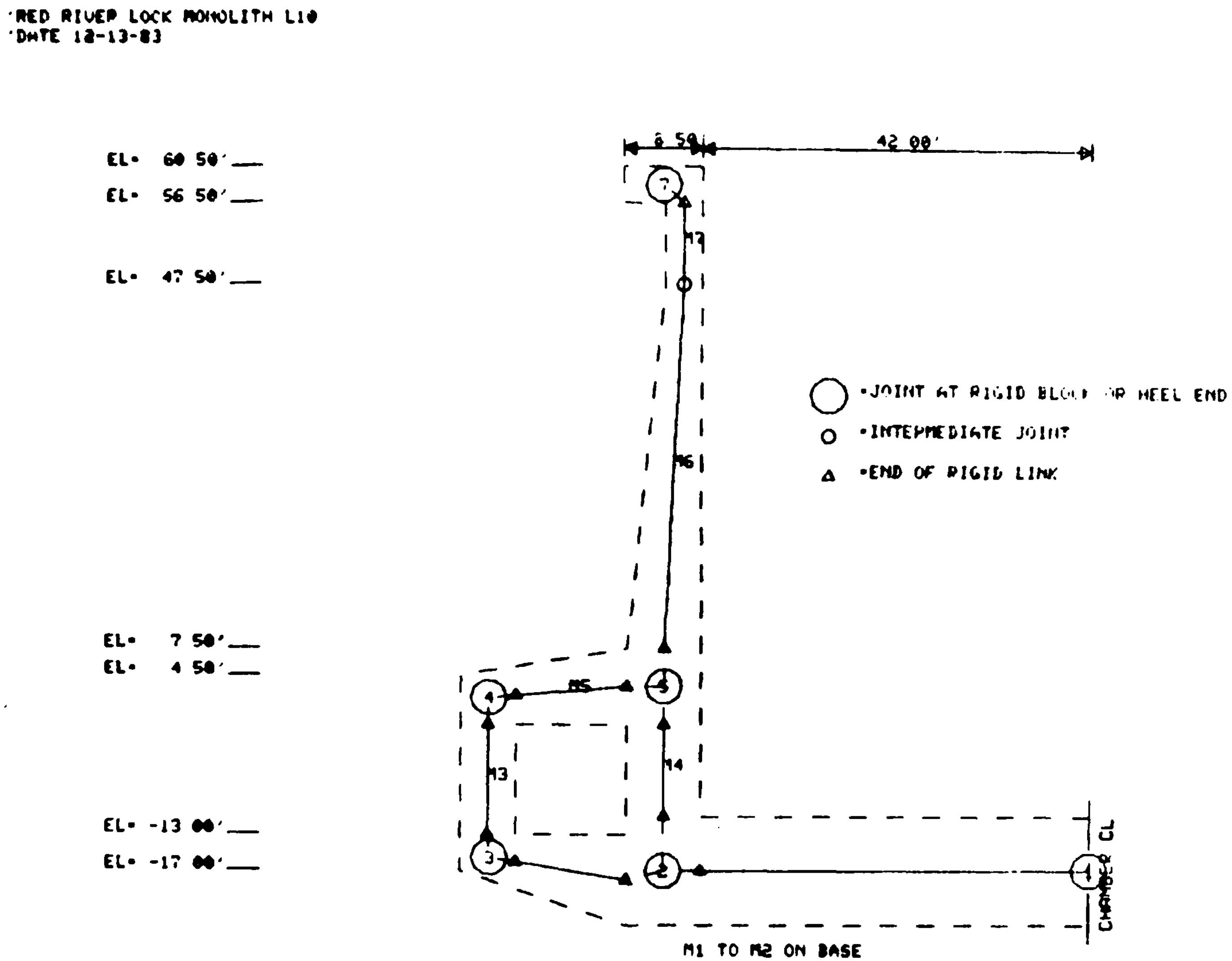
Riverside

Figure 24. CUFRAM input for riverside wall pressure on 6-11-85



a. Landside model

Figure 25. CUFRAM model for frame analysis (Continued)



b. Riverside model

Figure 25. (Concluded)

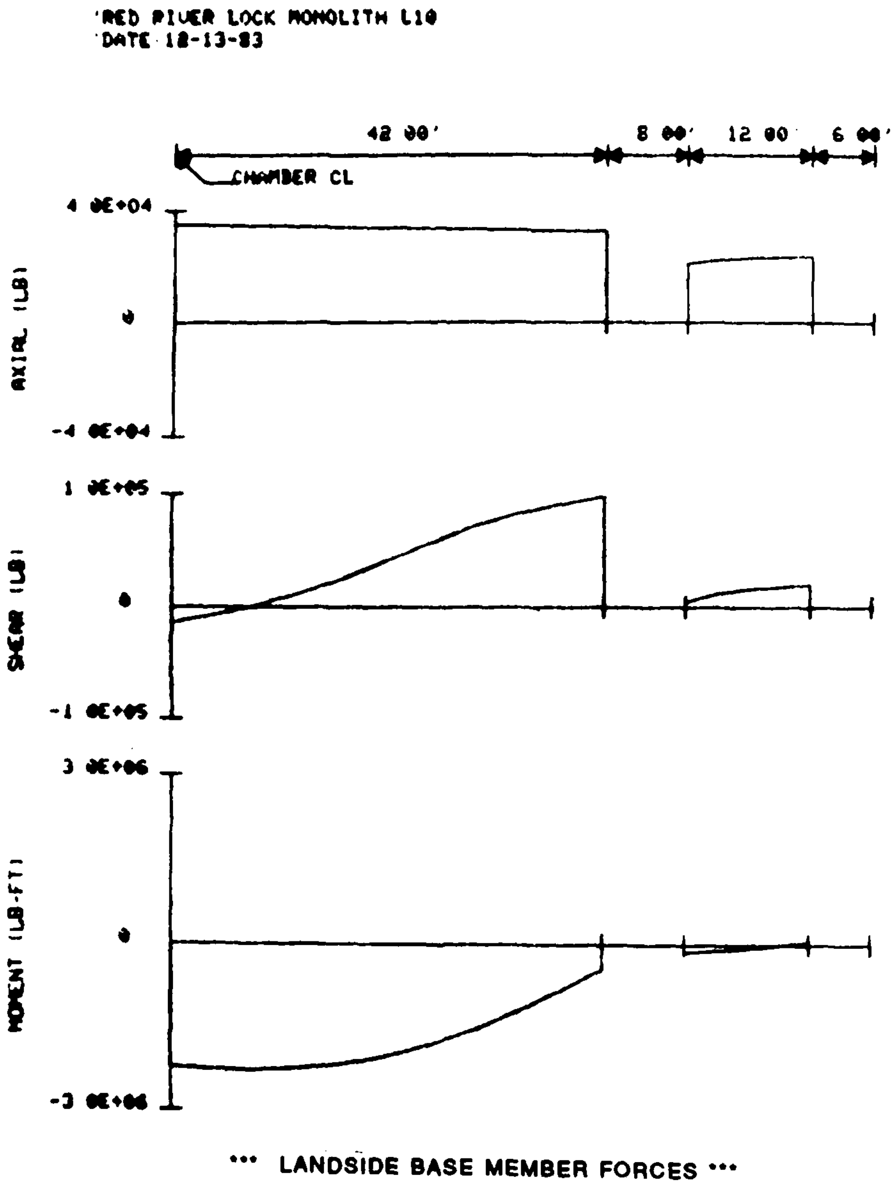


Figure 26. Internal forces for landside of base slab from CUFRAM analysis for 12-13-83

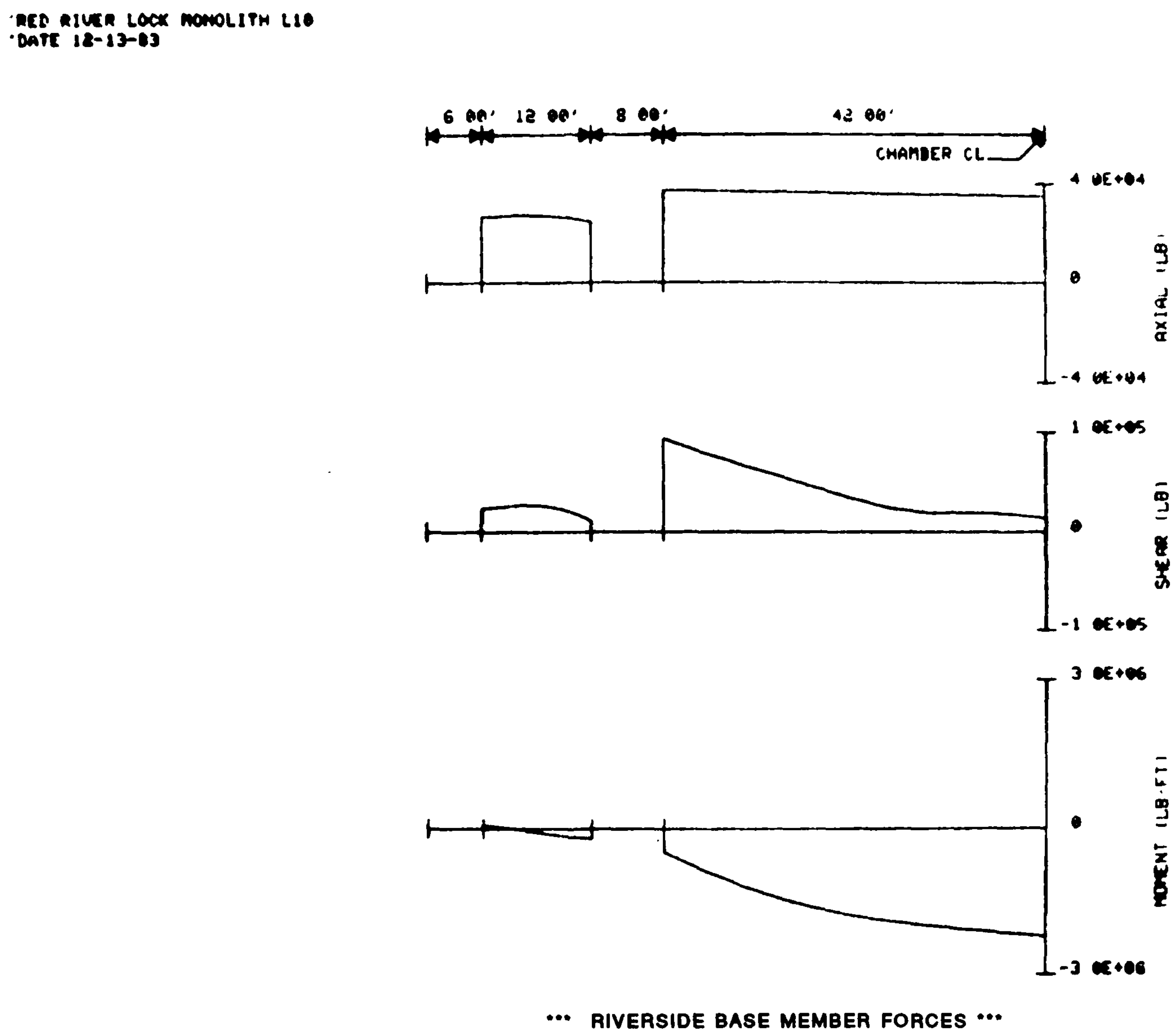


Figure 27. Internal forces for riverside of base slab from CUFRAM analysis for 12-13-83

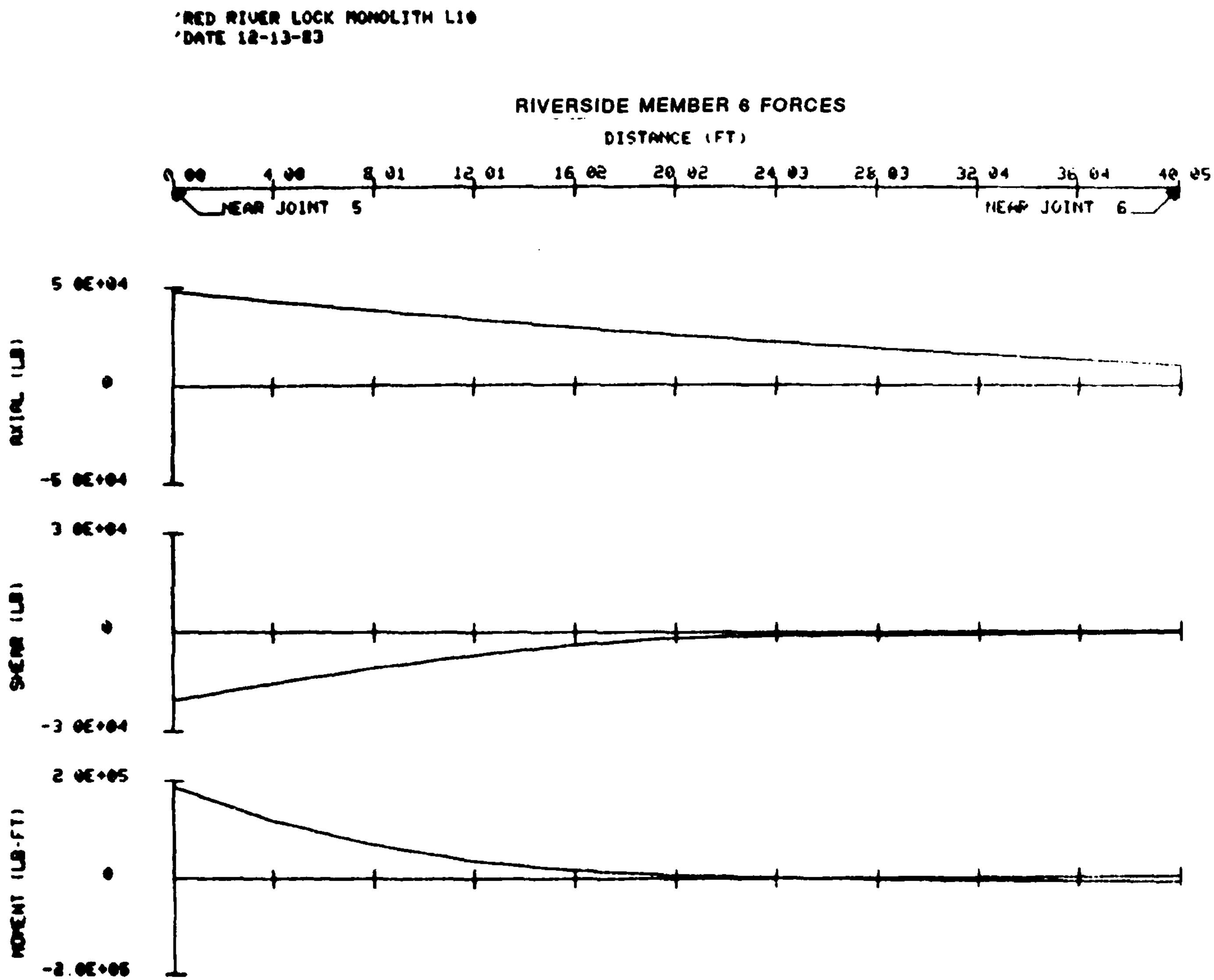


Figure 28. Internal forces for riverside stem wall from CUFRAM analysis for 12-13-83

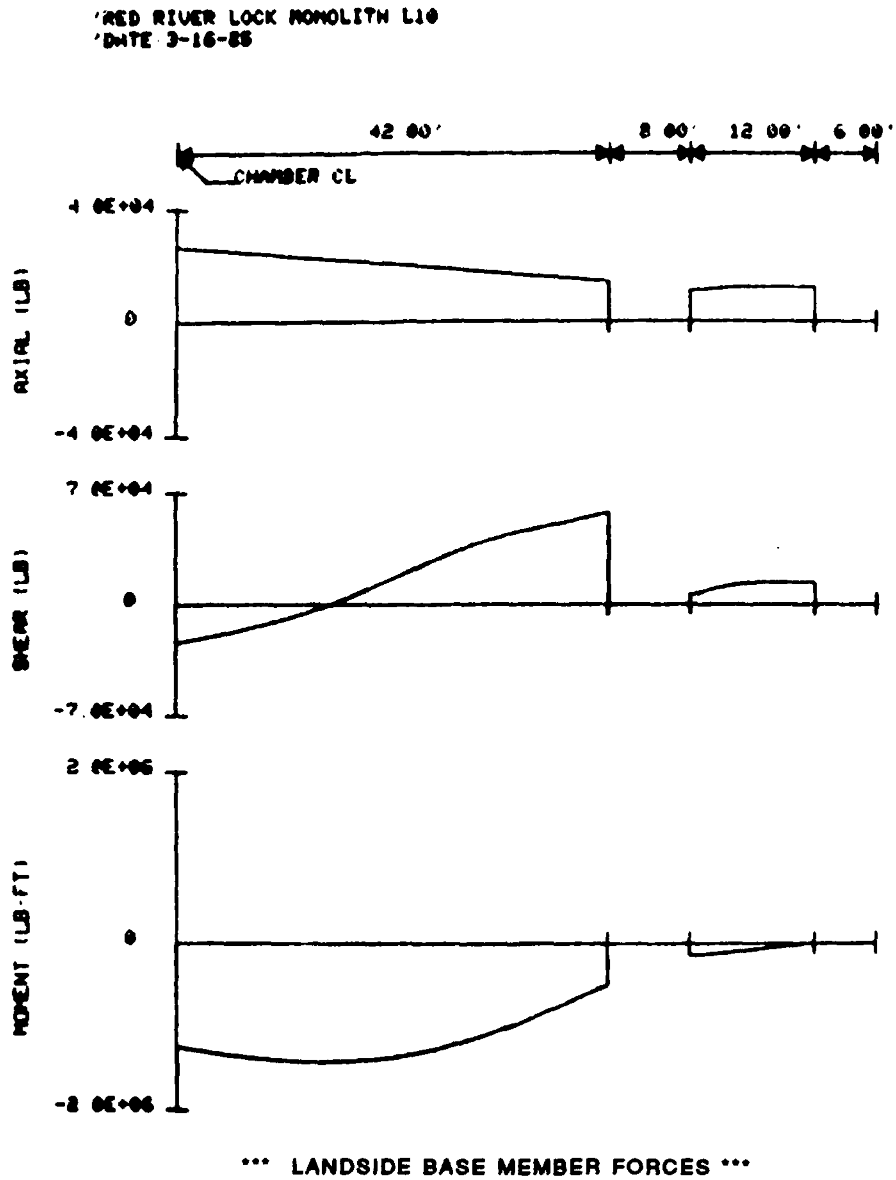


Figure 29. Internal forces for landside of base slab from CUFRAM analysis for 3-16-85

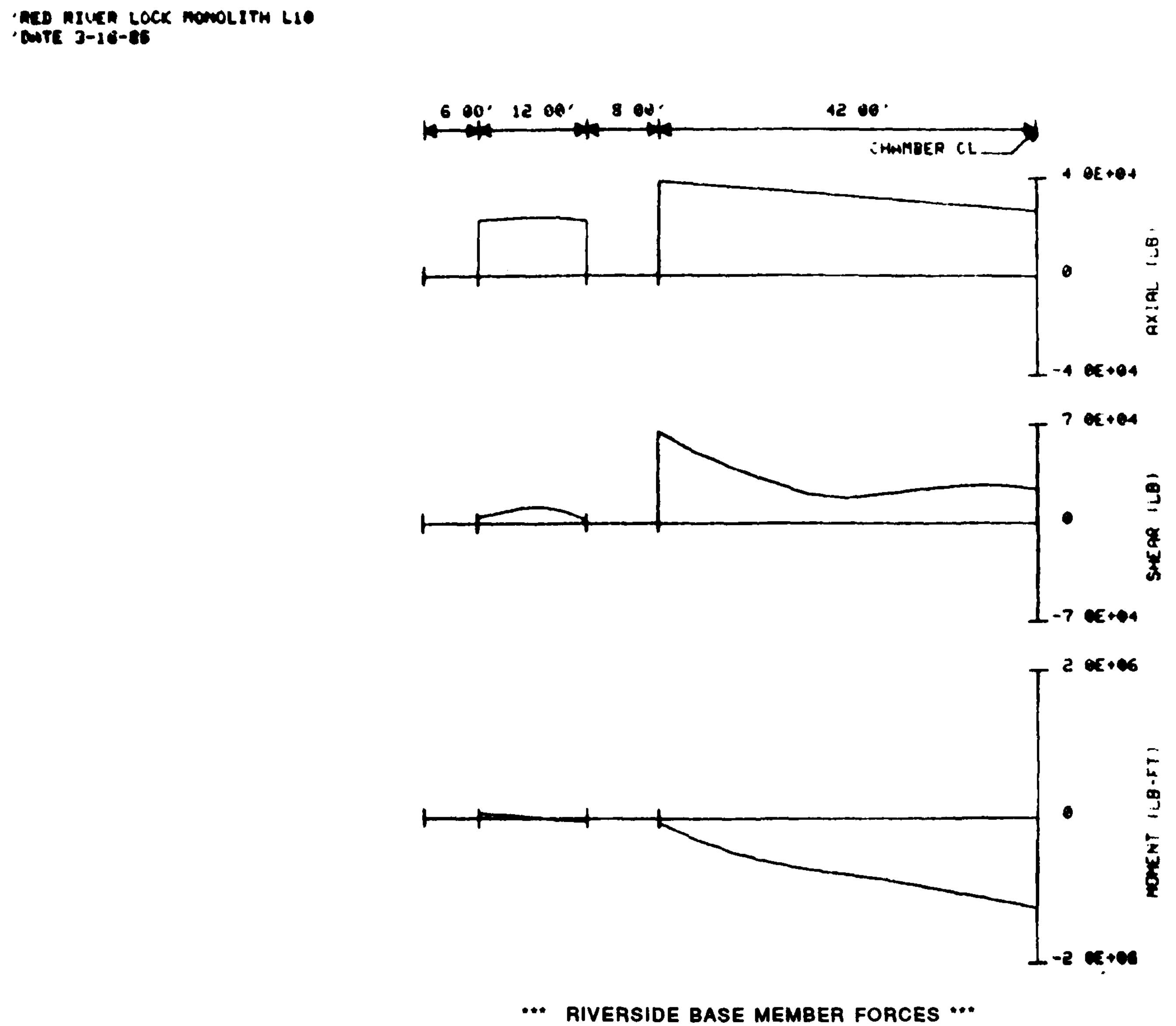


Figure 30. Internal forces for riverside of base slab from CUFRAM analysis for 3-16-85

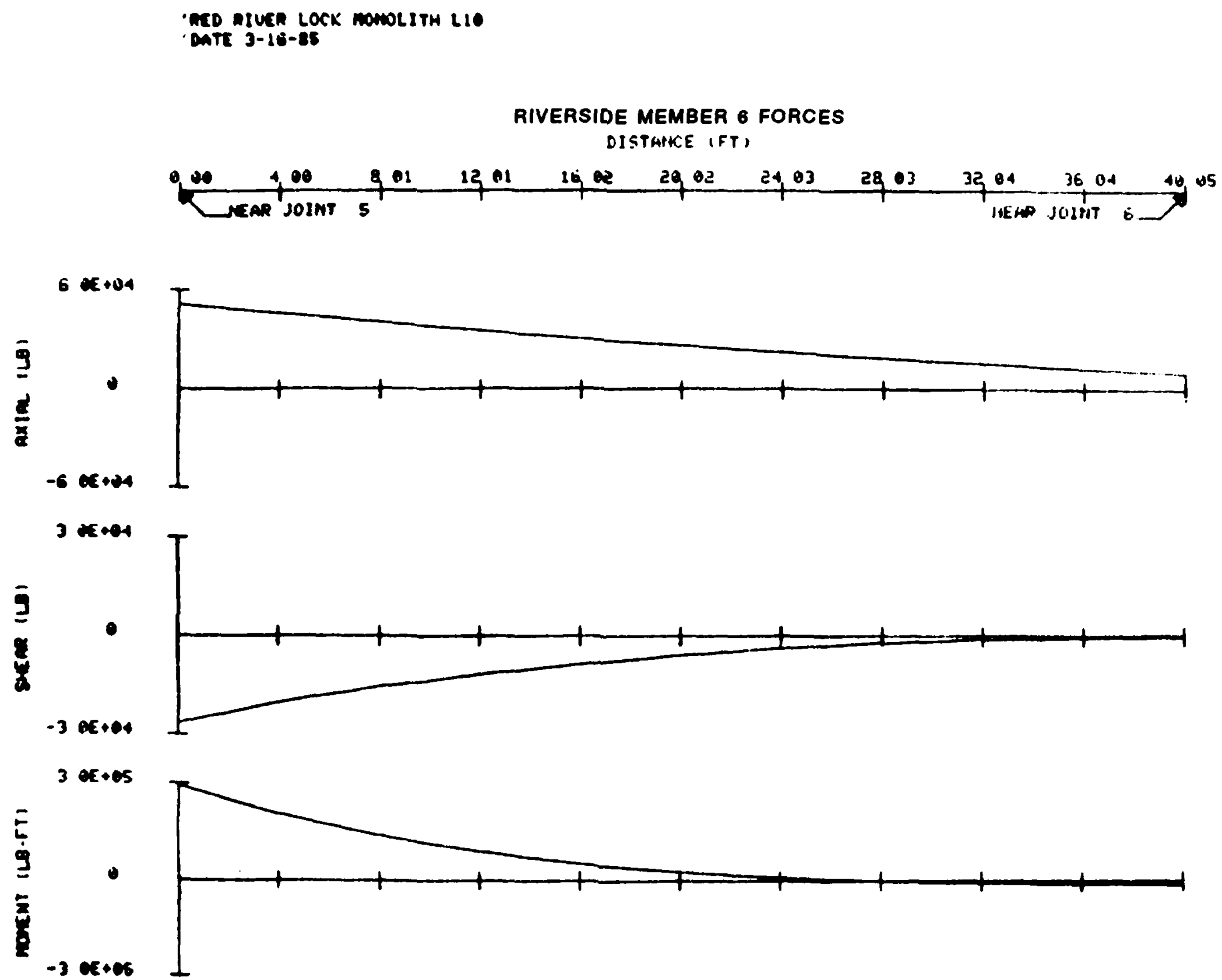


Figure 31. Internal forces for riverside stem wall from CUFRAM analysis for 3-16-85

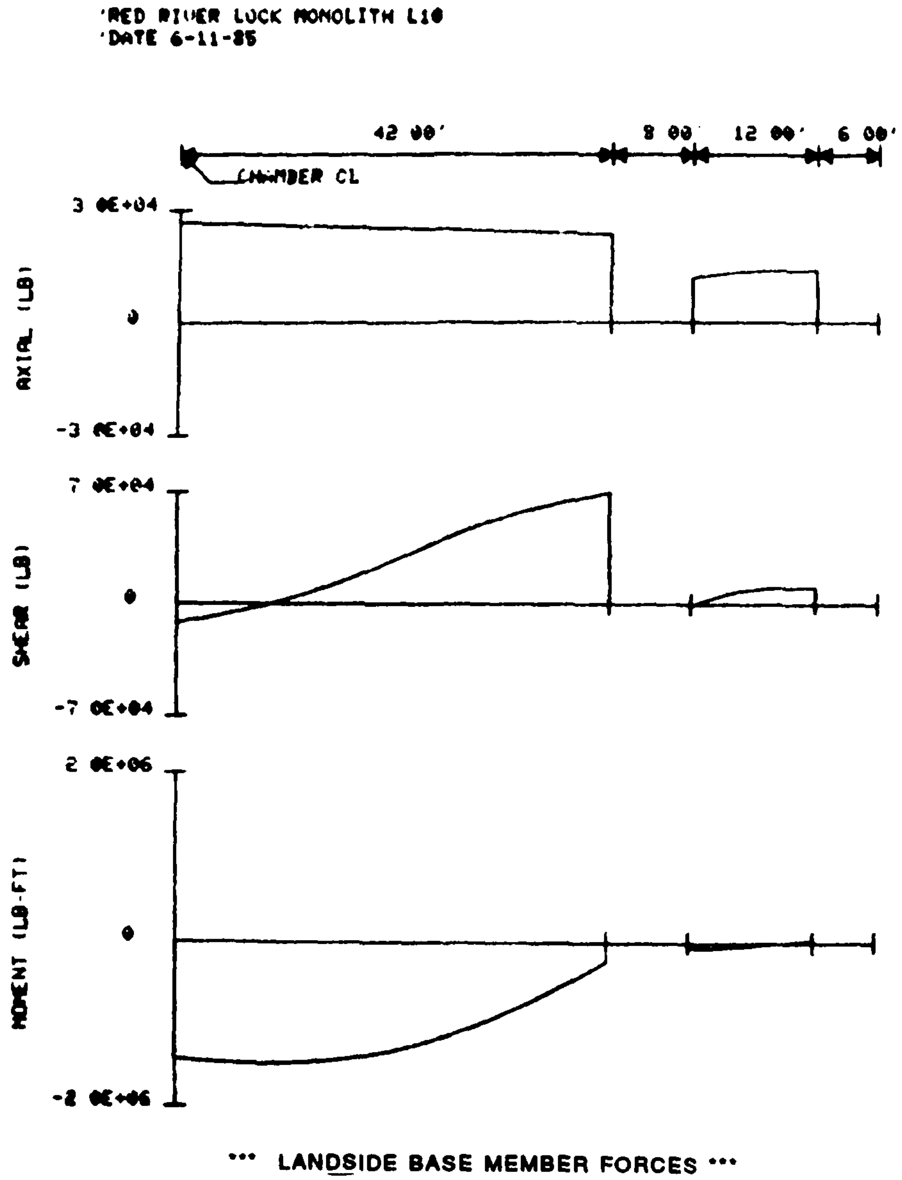


Figure 32. Internal forces for landside of base slab from CUFRAM analysis for 6-11-85

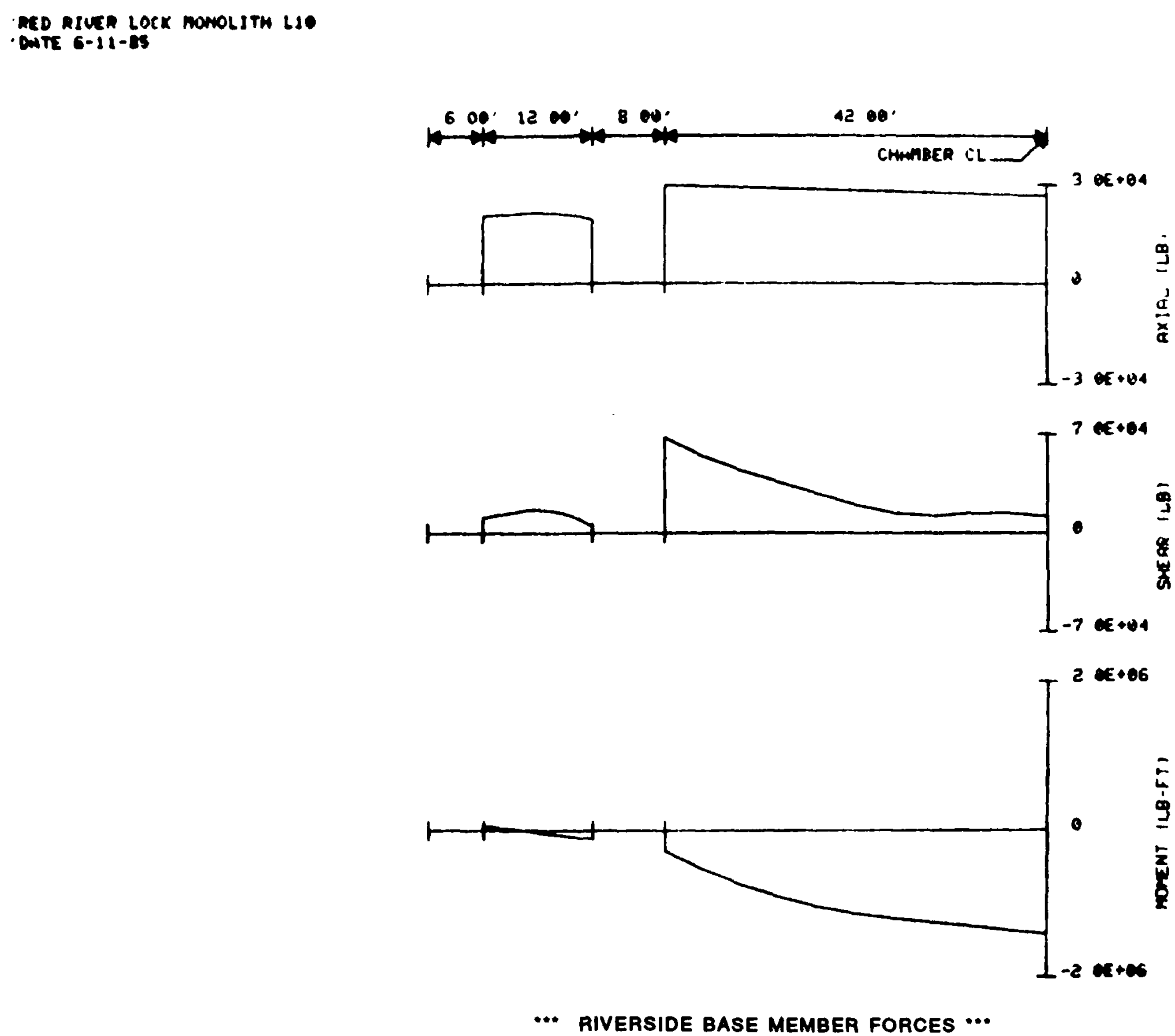


Figure 33. Internal forces for riverside of base slab from CUFRAM analysis for 6-11-85

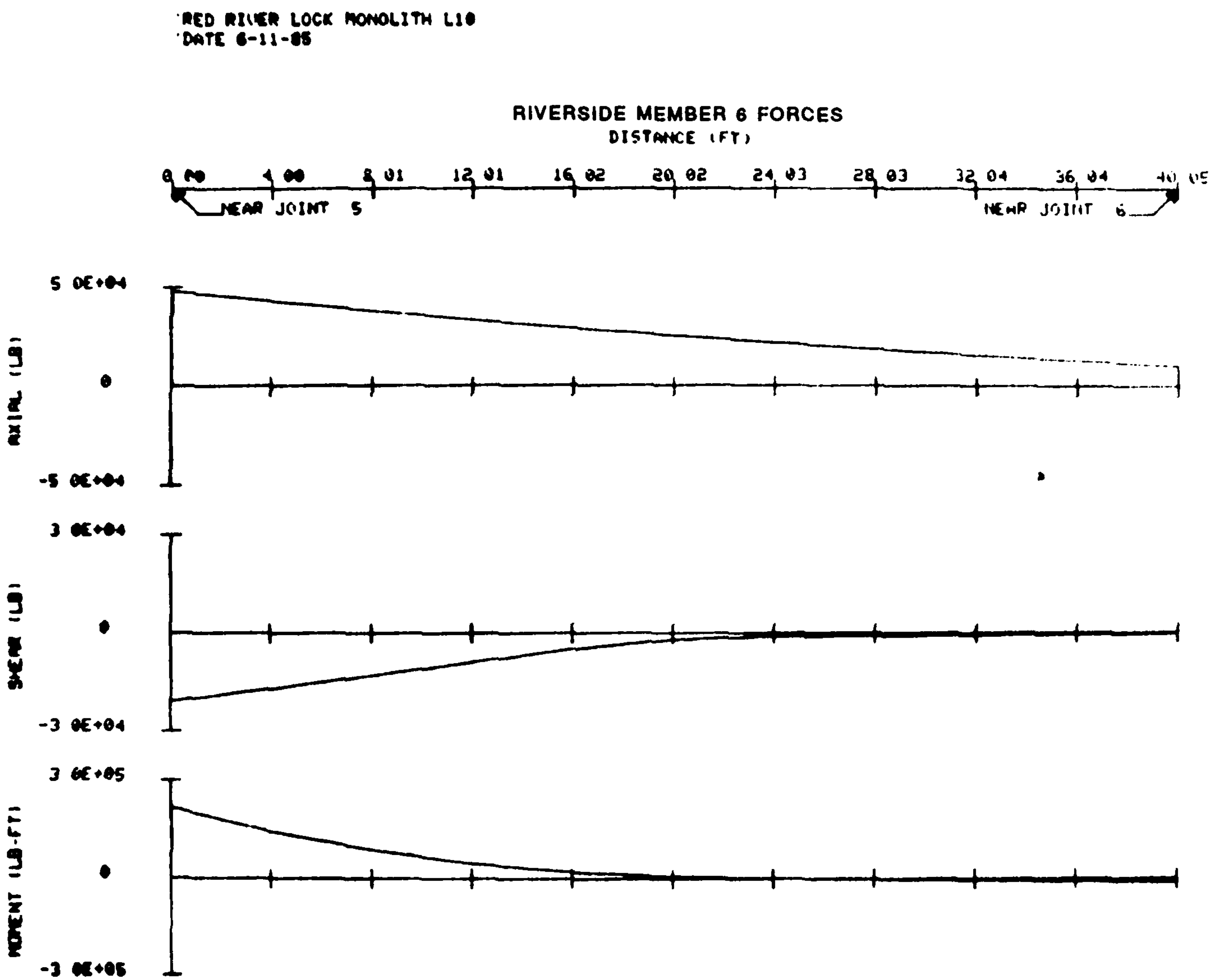


Figure 34. Internal forces for riverside stem wall from CUFRAM analysis for 6-11-85

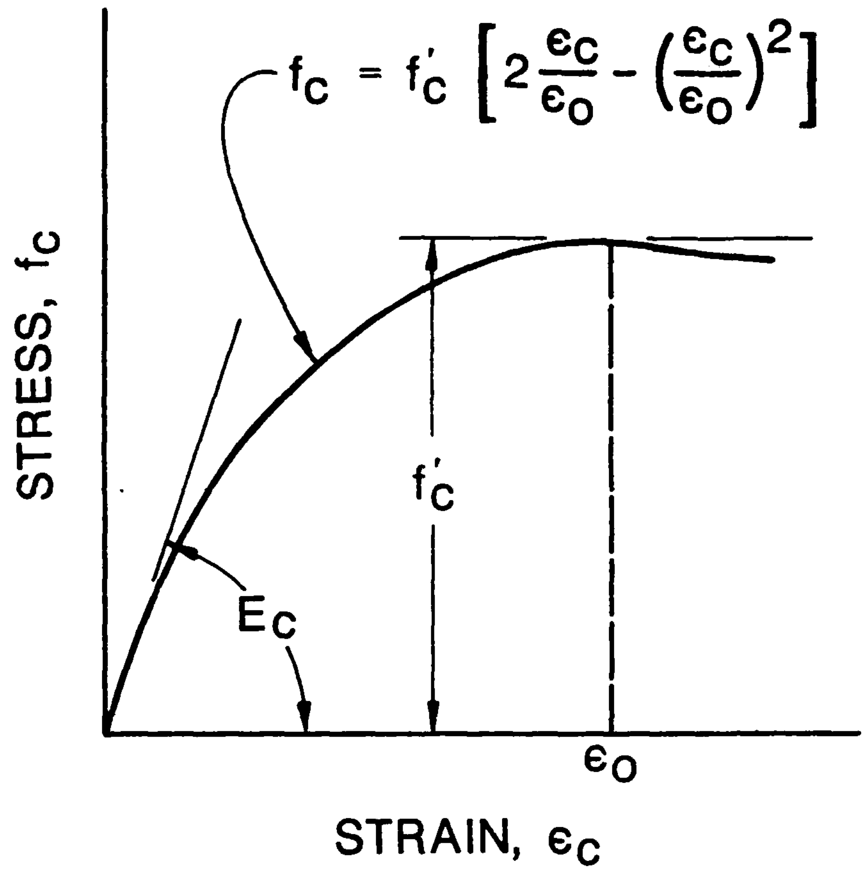


Figure 35. Concrete stress-strain function used for analyses for RSG data (Hognestad 1951)

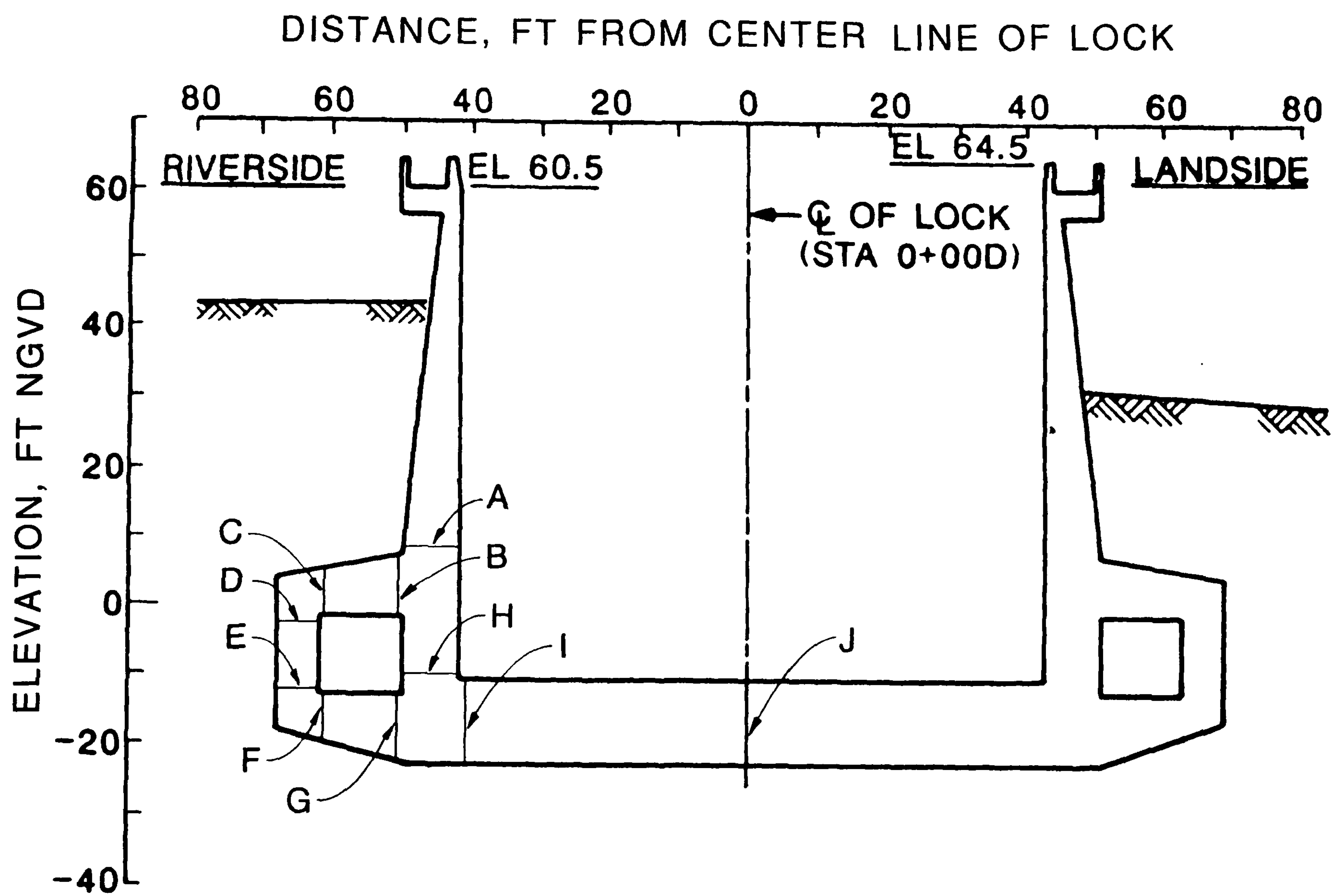


Figure 36. Critical sections for worst-case analysis